BASE ISOLATION
FOR
MULTISTOREY BUILDING STRUCTURES

A thesis
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Peng-Hsiang, Charng

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ABSTRACT

Earthquakes are one of nature's greatest hazards; throughout historic time they have caused significant loss of life and severe damage to property, especially to man-made structures. On the other hand, earthquakes provide architects and engineers with a number of important design criteria foreign to the normal design process. From well established procedures reviewed by many researchers, seismic isolation may be used to provide an effective solution for a wide range of seismic design problems.

The application of the base isolation techniques to protect structures against damage from earthquake attacks has been considered as one of the most effective approaches and has gained increasing acceptance during the last two decades. This is because base isolation limits the effects of the earthquake attack, a flexible base largely decoupling the structure from the ground motion, and the structural response accelerations are usually less than the ground acceleration.

In this research, a series of dynamic analyses are carried out to investigate in detail the seismic responses for stiff and flexible 12-storey multistorey buildings to the various isolation systems and to consider the effects of foundation compliance on their responses when subjected to different earthquakes. At the same time, an investigation of the seismic response of the recently suggested segmental buildings is carried out. The segmental building concept can be considered as an extension of the conventional base isolation technique with additional flexibility distributed in the superstructure. In addition to the conventional isolation system placed at the base, the superstructure of segmental buildings is further divided into several segments which are interconnected by extra isolation systems located in the upper storeys.

In general, the increase of additional viscous damping in the structure may reduce displacement and acceleration responses of the structure. This study also seeks to evaluate the effects of additional damping on the seismic response when compared with structures without additional damping for the different ground motions. In addition, analysis and design considerations for base isolated and segmental structures are suggested to enable the designer to get a better understanding at the preliminary design stage.

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In particular, I would like to sincerely dedicate this thesis to the memory of my mother.

TABLE OF CONTENTS

NOTATIONS

a_0	= dimensionless frequency
b	= maximum horizontal dimension of the building
c	= cohesion of soil
	= dimensionless dynamic damping coefficient of the vibration mode
c_{ce}	= frequency, shape and embedment front coefficient of horizontal damping
c_s	= frequency, shape and embedment side coefficient of horizontal damping
c_x	= frequency and shape dependent coefficient of horizontal damping in x-direction
c_y	= frequency and shape dependent coefficient of horizontal damping in y-direction
c_z	= frequency and shape dependent coefficient of vertical damping
e_b	= design eccentricity
g	= acceleration of gravity
h	= distance from the mid-height of the sidewall to the ground surface
h_i	= height of ith floor
i	= mode index
k	= dimensionless dynamic stiffness coefficient of the vibration mode
k_{eff}	= effective stiffness of base isolation system
k_{max}	= maximum effective stiffness of base isolation system
k_{min}	= minimum effective stiffness of base isolation system
k_o	= initial or elastic stiffness
$k_{v,emb}$	= dynamic vertical stiffness coefficient of embedded foundation
$k_{v,sur}$	= dynamic vertical stiffness coefficient of surface foundation
$k_{v,tre}$	= dynamic vertical stiffness coefficient of trench foundation
$k_{x,emb}$	= dynamic horizontal stiffness coefficient in x-direction
$k_{y,emb}$	= dynamic horizontal stiffness coefficient in y-direction
p	= exponent used in the Code-Type approach formula for predicting the equivalent lateral force distribution
r	= displacement of a degree-of-freedom due to a unit ground displacement
	= ratio of elastic stiffness to post-yield stiffness of the bilinear model
t	= time
u	= relative displacement
$\dot{u}$	= relative velocity
$\ddot{u}$	= relative acceleration

u_g	= ground displacement
$\dot{u}_g$	= ground velocity
$\ddot{u}_g$	= ground acceleration
v	= total-motion displacement
$\dot{v}$	= total-motion velocity
$\ddot{v}$	= total-motion acceleration
y	= total displacement of the system
y_0	= displacement of the lower layer of the system
y_1	= displacement of the upper layer of the system
A_b	= base area of footing foundation
A_s	= soil-sidewall contact area of embedded foundation
A_w	= effective soil-sidewall contact area of embedded foundation
A_{ws}	= soil contact area of sidewall which is parallel with horizontal motion
A_{wce}	= soil contact area of sidewall which is perpendicular with horizontal motion
B	= damping coefficient corresponds to the damping value in percentage of critical damping
	= one-half of footing width
C	= damping matrix
	= radiation damping coefficient
	= basic seismic coefficient (NZS 4203:1976)
	= lateral force coefficient (NZS 4203:1992)
C_b	= damping coefficient for a base isolation system
	= horizontal damping coefficient due to foundation base
C_{bx}	= horizontal damping coefficient due to foundation base in x-direction
C_{by}	= horizontal damping coefficient due to foundation base in y-direction
C_d	= seismic coefficient (NZS 4203:1976)
	= dynamic seismic coefficient
C_{eff}	= additional equivalent viscous damping coefficient
C_{eq}	= equivalent soil-foundation damping coefficient
$C_h(T_1, \mu)$	= basic seismic hazard acceleration coefficient
C_v	= vertical damping coefficient of soil-foundation
C_w	= horizontal damping coefficient due to sidewall
C_0	= damping coefficient for the lower layer of the soil-foundation
C_1	= damping coefficient for the upper layer of the soil-foundation
D	= depth of embedment

D	= minimum lateral seismic displacement
D_T	= total design displacement
D_{TM}	= total maximum displacement
E_h	= additional hysteretic damping of the base isolated structure
F_i	= equivalent static lateral force at ith floor
F_y	= yield strength or yield force
F_y^+	= maximum positive yield force
F_y^-	= maximum negative yield force
F^+	= maximum positive force
F^-	= maximum negative force
G	= soil shear modulus
G_{cm}	= complex soil shear modulus
G_{max}	= maximum soil shear modulus
G_0	= soil shear modulus at ground surface
G_1	= apparent soil shear modulus
I	= importance factor (NZS 4203:1976)
	= degree of isolation given by $T_b/T_1(U)$
I_{tre}	= coefficient of trench effect
I_{wall}	= coefficient of sidewall effect
K	= stiffness matrix
K_b	= stiffness of linear isolator
K_{b1}	= initial or elastic stiffness of bilinear isolator
K_{b2}	= post-yield or plastic stiffness of bilinear isolator
K_B	= effective or secant stiffness of bilinear isolator
K_d	= dynamic stiffness of the dynamic impedance of soil-foundation
K_s	= static stiffness of the dynamic impedance of soil-foundation
K_{eq}	= equivalent soil-foundation stiffness coefficient
K_0	= lower soil-found stiffness coefficient
K_1	= upper soil-found stiffness coefficient
$K_{dv,emb}$	= dynamic vertical stiffness of embedded foundation
$K_{sv,emb}$	= static vertical stiffness of embedded foundation
$K_{dx,emb}$	= dynamic horizontal stiffness of embedded foundation in x-direction
$K_{dy,emb}$	= dynamic horizontal stiffness of embedded foundation in y-direction
$K_{sx,emb}$	= static horizontal stiffness of embedded foundation in x-direction
$K_{sy,emb}$	= static horizontal stiffness of embedded foundation in y-direction

$K_{sv,sur}$	= static vertical stiffness of surface foundation
$K_{sx,sur}$	= static horizontal stiffness of surface foundation in x-direction
$K_{sy,sur}$	= static horizontal stiffness of surface foundation in y-direction
L	= one-half of foundation length
L_u	= limit state factor (NZS 4203:1992)
M	= structural material factor (NZS 4203:1976)
	= total mass of structure
	= mass matrix
N	= near-field coefficient proximity to active faults
NL	= non-linearity factor
P	= axial load
P_i	= applied load relating to the ith displacement mode
P(t)	= time varying loading on the structure
P_0	= axial load for lower layer of soil-foundation
P_1	= axial load for upper layer of soil-foundation
PF_i	= participation factor for ith mode
Q_y	= yield force at yield displacement
Q_y/W	= ratio of yield force to weight of bilinear isolator
R	= seismic risk factor (NZS 4203:1976 or NZS 4203:1992)
	= hysteretic loop ratio
R_w	= ductility factor of the superstructure for conventional structure
R_{wi}	= force reduction coefficient
S	= structural type factor (NZS 4203:1976)
	= site coefficient based on soil profile
$S_A(T, \zeta)$	= spectral absolute acceleration for period T and damping ζ
S_b	= maximum base shear
$S_D(T, \zeta)$	= spectral relative displacement for period T and damping ζ
S_p	= structural performance factor (NZS 4203:1992)
S_y	= shape dependent coefficient of soil-foundation horizontal stiffness
S_z	= vertical static stiffness parameter
T	= natural period of the isolated structure
T_b	= natural period of linear base isolator
T_{b1}	= period in elastic region of bilinear isolator
T_{b2}	= period in plastic region of bilinear isolator
T_B	= effective period for bilinear isolator

T_{eff}	= effective period of the structure
T_1	= fundamental period of vibration for the direction being considered
$T_{1\ eff}$	= effective fundamental period
$T_{1\ (U)}$	= unisolated undamped first mode period
$T_{1\ (UI)}$	= fundamental period of unisolated structure
V	= base shear
V_b	= minimum shear force below the isolation system
V_s	= minimum shear force above the isolation system
	= shear wave velocity
V_{ce}	= compression extension wave velocity
V_{La}	= Lysmer's analog wave velocity
$V_{s,\ m}$	= shear wave velocity taking into account of the effect of soil material damping
W	= total dead weight of the structure (NZS 4203:1976)
	= total weight of structure
W_d	= work done for the energy dissipated at the peak displacement
W_i	= portion of the total weight located at level i
	= total gravity load (NZS 4203:1976)
W_t	= total seismic weight of the structure (NZS 4203:1992)
X_b	= maximum relative base displacement of isolated structure
X_{max}	= maximum peak horizontal displacement
Y	= modal amplitude
Y_i	= modal amplitude at ith mode
Z	= seismic zone factor (NZS 4203:1992)
α	= ratio of elastic stiffness to post-yield stiffness of the bilinear model
	= a constant related to the mass matrix in Rayleigh's damping model
β	= a constant related to the stiffness matrix in Rayleigh's damping model
Δ_{max}	= maximum design displacement
Δ^+	= corresponding maximum positive test displacement
Δ^-	= corresponding maximum negative test displacement
ζ_b	= velocity-damping factor for isolator
ζ_{b2}	= velocity-damping factor in plastic region of bilinear isolator
ζ_B	= effective damping factor of bilinear isolator
ζ_h	= hysteretic damping factor of bilinear isolator
ζ_v	= viscous damping factor of bilinear isolator
λ	= minimum effective damping of the base isolated structure

$\lambda_{add.}$	= additional hysteretic damping of the base isolated structure
λ_{eff}	= additional damping ratio
$\lambda_{1\,eff}$	= effective equivalent viscous damping of mode 1
μ	= ratio of maximum displacement to the yield displacement of base isolation system
	= structural ductility (NZS 4203:1992)
ν	= soil Poisson's ratio
ξ_m	= damping ratio effect of soil material damping
π	= 3.1415926
ρ	= soil mass density
ϕ	= angle of internal friction
	= modal matrix
$\{\phi\}_i$	= ith mode of free vibration
ω	= natural frequency of structure
ω_{eff}	= effective circular frequency of the structure
ω_i	= ith frequency of free vibration

ABBREVIATIONS

ATC	= Applied Technology Council
DIS	= Dynamic Isolation Systems
EERI	= Earthquake Engineering Research Institute
NZNSEE	= New Zealand National Society for Earthquake Engineering
PTFE	= Polytetrafluoroethylene
RMS	= Root-Mean-Square
SEAOC	= Structural Engineers Association of California
SEAONC	= Structural Engineers Association of Northern California
SRSS	= Square Root of the Sum of the Squares
UBC	= Uniform Building Code

CHAPTER 1

INTRODUCTION

1.1 General

Recent earthquakes, particularly the 1989 Loma Prieta [E2] and 1994 Northridge [E3] earthquakes in California, and the 1995 Kobe [K1] earthquake in Japan, have caused significant loss of life and severe damage to property. Many aseismic construction designs and technologies have been developed over the years in attempts to mitigate the effects of earthquakes on buildings and their vulnerable contents. Attenuating the effects of severe ground motions on the buildings and their contents is always one of the most popular topics in the area of civil and structural engineering and attracts the attention of many researchers and engineers around the world.

The technique of base isolation has been developed in an attempt to mitigate the effects on buildings and their contents during earthquake attacks and has been proven to be one of the more effective methods for a wide range of seismic design problems on buildings in the past two decades. Seismic isolation consists essentially of the installation of mechanisms which decouple the structures and their contents from potentially damaging earthquake-induced ground motions. This decoupling is achieved by increasing the flexibility of the systems, together with providing appropriate damping. Careful studies have been made of structures for which seismic isolation may find widespread application. This has been found to include common forms of new and existing multistorey building structures.

In seismic isolation, the fundamental aim is to reduce substantially the transmission of the earthquake forces and energy into the structure. This is achieved by mounting the structure on an isolation system with considerable horizontal flexibility so that during an earthquake, when the ground vibrates strongly under the structure, only moderate motions are induced within the structure itself. As the isolator flexibility increases, movements of the structure relative to the ground may become a problem under other vibrational loads applied above the level of the isolator, particularly wind loads. Skinner et al (1993) [S6] indicated that a base isolator with hysteretic force-displacement characteristics can provide the desired properties of isolator flexibility, high damping and force limitation under horizontal earthquake loads, together with high stiffness under smaller horizontal loads to limit wind-induced motions.

Kelly (1990) [K2] gave a brief introduction to the response mechanisms of base isolated buildings through a two degrees of freedom linear system. The effectiveness of the isolation system to mitigate the seismic response is through its ability to shift the fundamental frequency of the system out of the range of frequencies where the earthquake is strongest. Also, Skinner et al (1993) demonstrated that the most important feature of seismic isolation is that its increased flexibility increases the natural period of the structure. Because the period is increased beyond that of the earthquake, resonance is avoided and the seismic acceleration response is reduced.

The successful base isolation of a building depends on the installation of mechanisms which decouple the structure from potentially damaging earthquake-induced ground motions. Therefore, it is very important to have an adequate understanding of the influence of each parameter in the isolation system and the superstructure on the seismic performance of the base isolated buildings. The primary function of an isolation device is to support the superstructure while providing a high degree of horizontal flexibility. This gives the overall structure a long effective period and hence lower earthquake generated accelerations and inertia forces. Many kinds of isolation systems have been developed to achieve this function, such as laminated elastomeric rubber bearings, lead-rubber bearings, yielding steel devices, friction devices (PTFE sliding bearings) and lead extrusion devices, etc..

Andriono (1990) [A2] indicated that base isolated structures have the ability to significantly reduce the ductility demands in the superstructure compared with those of unisolated structures. This makes possible simplification of the structural detailing and other seismic design considerations required by the more conventional approaches. Therefore, a wider choice of architectural forms and structural materials is available to the designer.

Besides the technical feasibility, another key issue that must be addressed early in the design phase of a base isolated building is the economic feasibility. In terms of economic cost, four principal factors should be evaluated. Kelly (1990) indicated that these are construction costs, earthquake insurance premiums, physical damage for repair and disruption costs, loss of market share and potential liability. Skinner et al (1993) showed that the current practice for seismic isolation techniques may often reduce the cost of providing a given level of earthquake resistance to buildings.

In the past two decades, base isolation has become an increasingly accepted technique for providing earthquake protection to buildings and their contents. On the other hand, base isolation has often been considered as a technique for problem structures or for equipment which requires a special seismic design approach. This may arise because of their function (sensitive or high risk industrial or commercial facilities such as computer systems or nuclear power plants); their special importance after an earthquake (hospitals, disaster control centres such as police stations); poor ground conditions (proximity to a major fault); or other special problems (increasing the seismic resistance of existing structures) as described by Skinner et al (1993).

Therefore, seismic isolation does indeed have particular advantages over other approaches in these special circumstances, usually being able to provide much better protection under extreme earthquake motions. It is believed that seismic isolation may be used to provide effective solutions for a wide range of seismic design problems.

1.2 Objectives of the Research

Several practical techniques for achieving seismic isolation and a variety of energy-dissipating devices have been developed and implemented around the world in recent years. Most practical work still relies upon a series of deterministic dynamic inelastic time history analyses [B5,C5]. The primary aim of this research is to give a better understanding of the behaviour of the base isolated buildings and a greater confidence in the behaviour of substructures under the most credible ground excitation. On the other hand, the results of analysing buildings using time history analyses subjected to the different earthquakes will be compared with those of the buildings designed to excitation associated with the response spectra found in New Zealand and other overseas building codes.

The first objective of this research is to investigate the seismic responses for stiff and flexible 12-storey buildings designed to New Zealand Standard Code of Practice for Design of Concrete Structures NZS 3101:1982 [N1] and NZS 3101:1995 [S9] to the different isolation systems and to consider the effects of foundation compliance during the different earthquakes.

Also, it seeks to evaluate the effect of using a segmental building proposed by Cui (1995) [C13] where the isolation devices are placed at various levels in the building in order to reduce the displacements imposed on each of the devices.

Usually the increase of added viscous damping in a structure may reduce the displacement and acceleration responses of the structure. Thus, the second objective of this research is to ascertain the effects of added viscous damping based on the equivalent static method recommended by New Zealand Standard Code of Practice for General Structural Design and Design Loadings for Buildings NZS 4203:1992 [S8] and then to investigate the seismic responses of the stiff and flexible buildings which will be compared with those of the structures without additional damping for different earthquake motions.

1.3 Scope and Outline of the Thesis

This research seeks to present a development of theoretical and analytical aspects of the behaviour of base isolated buildings. The following outline describes the scope of the study. To evaluate the development, the results and the proper selection for further research on base isolated structures, a review of the current design methods and existing design codes are presented in Chapter 2. The principles of dynamic analysis, structure modelling and soil-footing foundation impedance are discussed in Chapter 3. The procedures of the analyses of the base isolated structures are outlined in Chapter 4. This chapter involves soil site model related, comparison of earthquake and wind loads, selection of base isolators, dynamic parameters of the base isolated buildings, earthquake excitations and the methods used in the analyses.

Chapter 5 presents the seismic responses of base isolated buildings subjected to the El Centro N-S earthquake which is commonly considered as a standard and on which many codes have been historically based. In this chapter, the seismic responses of base isolated structures with elasto-plastic and bilinear isolation devices are investigated and compared with those of other types of structures such as fixed base and segmental buildings. Some discussion of curvature ductility demands of beams and columns in the frames are also presented.

The seismic responses of different types of structures including base isolated, fixed base and segmental buildings with added damping devices subjected to El Centro N-S earthquake are presented in Chapter 6. This chapter includes equivalent viscous damping calculated by the loading time history, dynamic responses of different structures with additional damping and comparison with those of structures without added damping and some discussion of curvature ductility demands of beams and columns for different types of buildings.

Effects of the ground motion characteristics on the structural behaviour of base isolated multistorey buildings are studied in order to be able to select the right system for a particular type of ground motion so that the base isolation device will provide a guaranteed benefit. A range of earthquake records other than the N-S component of the El Centro 1940 are used as the basis of these analyses. The chosen earthquake records were scaled according to their 5% damped spectra to fit the design spectrum in Section 4.6.2.9 (b) (ii) of NZS 4203:1992 code for the intermediate soil sites. The results of this investigation are reported in Chapter 7.

As discussed in chapters 5, 6 and 7, the buildings used were based on two different design standards. One building was designed to NZS 3101:1982 and the second one was designed to NZS 3101:1995, and in accordance with the provisions of loading code NZS 4203:1992.

Chapter 8 presents a consideration for the analysis and design of the base isolated and segmental buildings. In this chapter, a basic concept of the preliminary design choice enables the designer a better understanding of the design for the base isolated and segmental buildings. This chapter includes a proposed design procedure and example to explain the design of the base isolated and segmental structures.

Finally, some conclusions and recommendations for further research on base isolated and segmental multistorey buildings are presented in Chapter 9.

CHAPTER 2

REVIEW OF CURRENT DESIGN METHODS AND CODES FOR BASE ISOLATED STRUCTURES

2.1 Introduction

Many studies since the early 1970s have discussed and considered seismic isolation devices of various types of structures. These include performance of structures with their isolation systems in earthquakes, various kinds of isolation systems and the experimental studies on their performance, earthquake simulator tests of model structures mounted on isolation devices and computer studies of the response of simulated structures with isolation systems. Seismic base isolation reduces the response accelerations at the expense of relatively large base displacements when compared to conventional buildings and the analytical model of the isolation system plays an important role in the dynamic analysis of base isolated buildings, as it has a significant influence on the accuracy of the analysis results.

The concept of introducing isolation systems to mitigate seismic effects is a well-known technique and the development of many practical base isolation devices was accompanied by proposed design methods for base isolated structures. An equivalent linear analysis was used by most proposed design methods for approximating the inelastic behaviour of the base isolation systems as it affects the response of the elastic superstructure. The main purpose of these methods was to assist the designers to design the isolated structures without depending on a series of deterministic inelastic time history analyses. In the following, several design methods and codes are reviewed from the numerous designs of base isolation systems.

2.2 Design Methods

2.2.1 Priestley, Crosbie and Carr (1977) [C12,P7]

This research work was carried out to study the seismic performance of four, eight and twelve storey masonry shear walls mounted on base isolation devices. The main purpose of this study was to examine an alternative method of limiting the inertia forces in masonry buildings by seismic base isolation and also gave a significant contribution towards later attempts made to investigate the seismic response of base isolated multistorey structures. They used a series of deterministic time history analyses with earthquakes El Centro 1940 N-S, Taft 1952 N69W, and two artificial A1 and B1 accelerograms generated by Jennings [J1].

An equal-acceleration approximation for floor masses as suggested earlier by Skinner and McVerry [S4] using a single degree of freedom model was found to be inadequate in this research work. The distribution of the maximum base shear in proportion to floor mass resulted in a severe underestimation of the required moment capacity of the four and eight storey masonry walls, and produced an envelope for the twelve storey wall which is conservative near the base of the wall but non-conservative higher up the wall due to the influence of higher mode effects. Priestley et al [C12,P7] proposed a tentative design recommendation regarding the lateral inertia force distribution as follows:

1. The design lateral force should be found by distributing the base shear force V in accordance with NZS 4203:1976 [C9], as expressed in Eq. 2.1, with an additional 0.2V applied at the roof level to cover the inertia force distribution resulting from higher modes.

$$F_i = V \frac{W_i\, h_i}{\Sigma\, W_i\, h_i} \qquad (2.1)$$

where F_i is the equivalent static lateral force at ith floor, W_i and h_i are the total gravity load and the height of ith floor respectively. An additional 0.2V to the roof storey effectively increased the shear envelope at all levels by 20% of the base shear.

$$V = C_d\, W \qquad (2.2)$$

in which

$$C_d = CISMR \qquad (2.3)$$

where V is the base shear, W is the total dead weight of the building, C,I,S,M,R are the basic seismic coefficient, the importance factor, the structural type factor, the structural material factor and the seismic risk factor respectively. In general, this would imply C_d equal to C based on the fact that S and M may both be set equal to unity because structural yield is avoided.

2. Crosbie [C12] proposed some reduced values of C, for each type of base isolation system considered, which should be used for determining the design forces in base isolated masonry structures. This was based on the results obtained from the above mentioned series of time history analyses. For example, the reduction of the basic seismic coefficient C from 0.288g to 0.160g would be recommended for the design of squat masonry shear walls mounted on base isolation systems with lead energy dissipators and located in the seismic Zone A [C9].

It should be emphasized that the above recommendations were based on a limited case study for certain types of structure and base isolation devices. It was found that this design requirement would provide an adequate flexural and shear capacity for short to intermediate period masonry shear walls (fundamental period less than 1.0 second). Further, it did not explicitly show the correlation between the capacity of the base isolation system in providing lateral flexibility and additional hysteretic damping and the structural response. Such correlation is essential to give the designers a clear understanding of how the base isolation system reduces the seismic forces within the structure.

2.2.2 DIS, Inc.'s Design Procedures for Buildings Mounted on Lead-Rubber Bearings (1984) [D5,M2]

These design procedures for buildings mounted on lead-rubber bearings were developed by a California based consultancy firm, Dynamic Isolation Systems (DIS), Inc.. The design procedures using a series of charts have been developed for ATC-3 [A5] seismic region maximum credible earthquakes with acceleration coefficients in the range of 0.20g to 0.40g for all soil types. Also included, are curves based on the Caltrans design spectra. The procedures are based on a single-degree-of-freedom representation of the building. It was stated in this design procedure that provided the period of the non-isolated building is less than 1.5 seconds and the building is reasonably symmetric, the single-degree-of-freedom assumption is a good approximation for design purposes.

The inelastic response of a multistorey building is approximately predicted in these design procedures by the pseudo elastic response of its fundamental mode. No specific guidance was given for the lateral force distribution up the height of the superstructure. Some modification had been conducted to transform these inelastic response spectra approaches into a number of design charts in a format considered by DIS as suitable for design use. Fig. 2.1 shows a flow chart to illustrate the step-by-step design method and Fig. 2.2 shows charts of each series used in this design procedure.

1. Assemble required data.

   Summarize the bearing location, the dead load and the dead plus live load for each bearing position. The dead load should include seismic live load.

2. Select plan dimension.

   From chart series 1 select the appropriate curve for the selected plan shape and internal rubber layer thickness.

3. Select rubber thickness.

   Using the total dead load and total area of all bearings compute the average compressive stress on all bearings. Select the appropriate graph from chart series 2 for the site acceleration level and soil conditions. Select a rubber thickness appropriate to the degree of isolation desired. Obtain the force coefficient and displacement by interpolation between the curves provided.

4. Check load capacity.

   Select the appropriate graph from chart series 3 for the rubber layer thickness and plan shape selected. For the bearing dimension adopted and the maximum

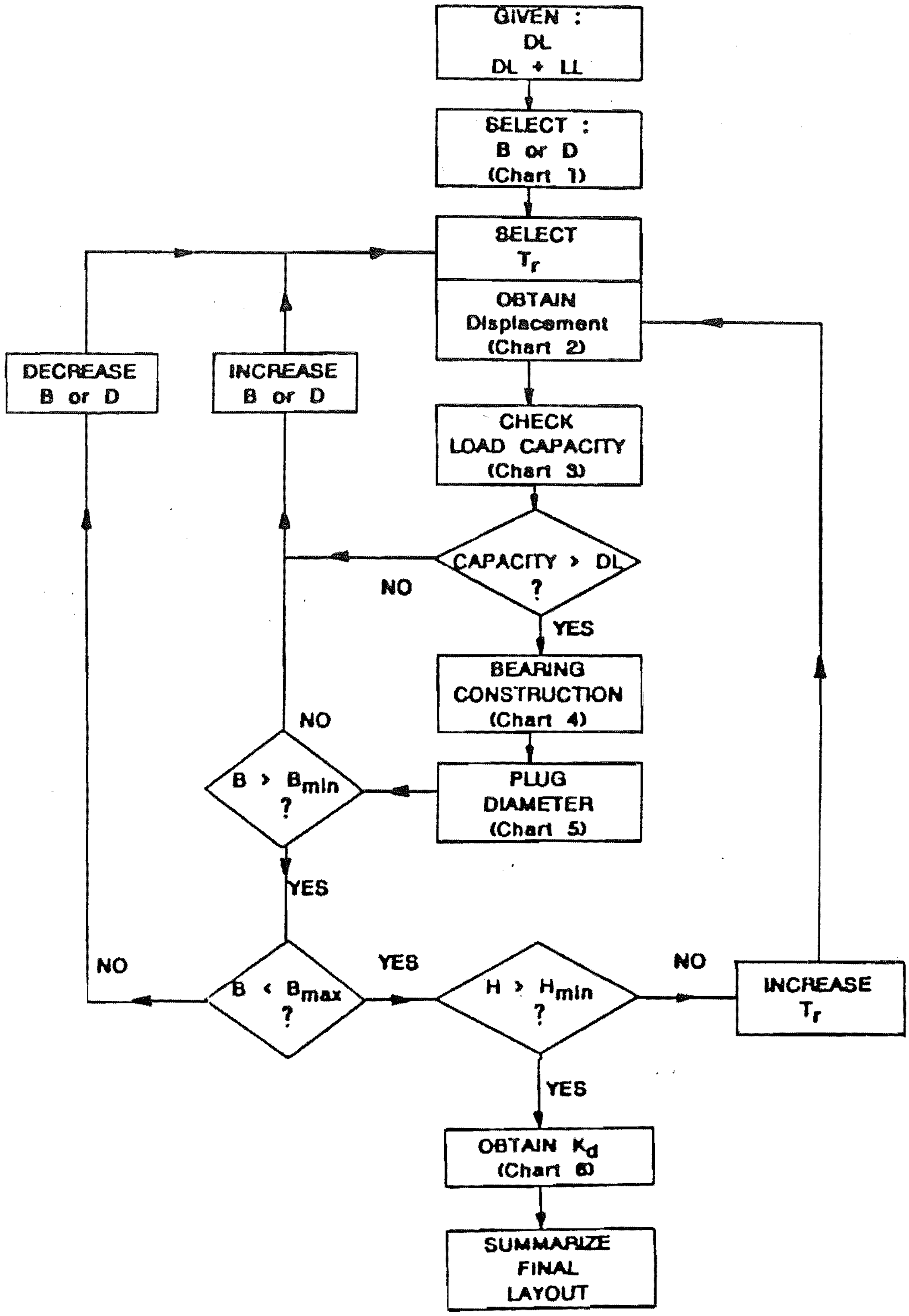


Fig. 2.1 DIS Lead Rubber Bearing Design Procedure

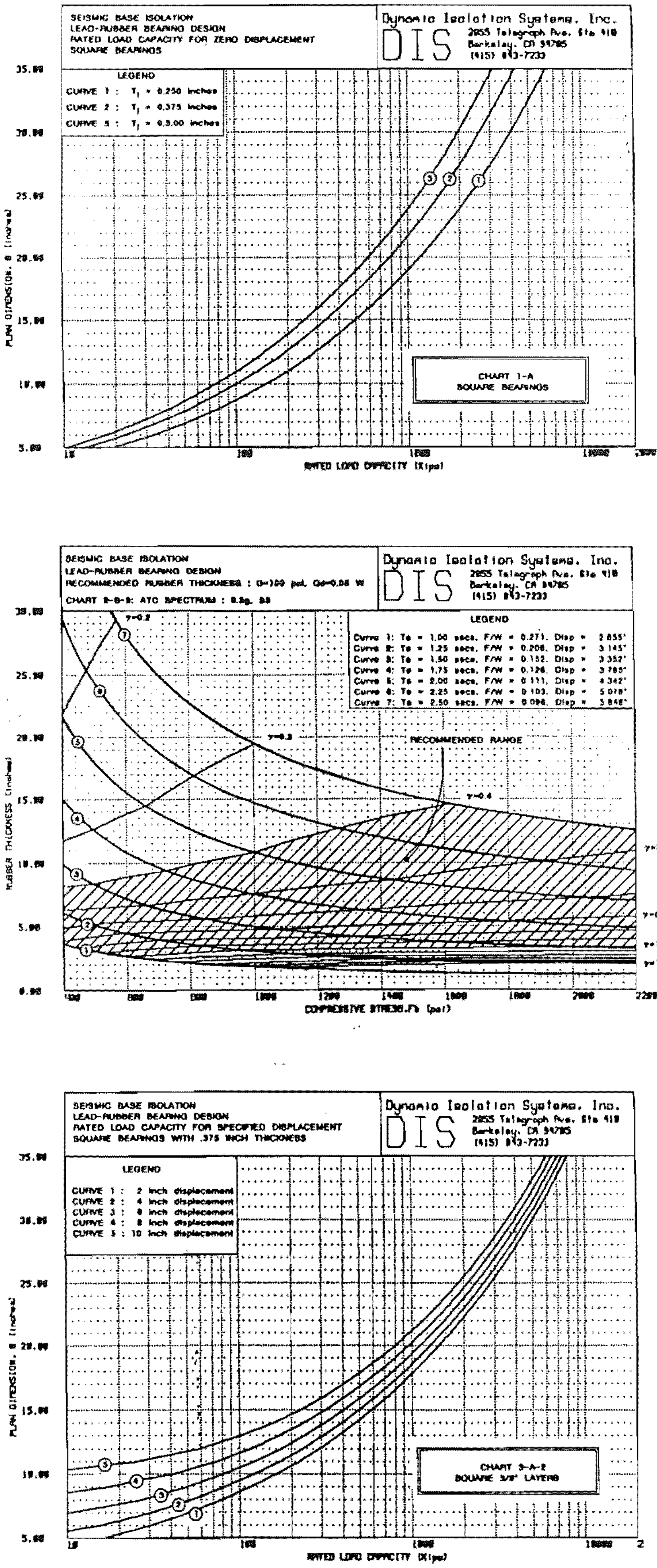


Fig. 2.2 The Series of DIS Design Charts

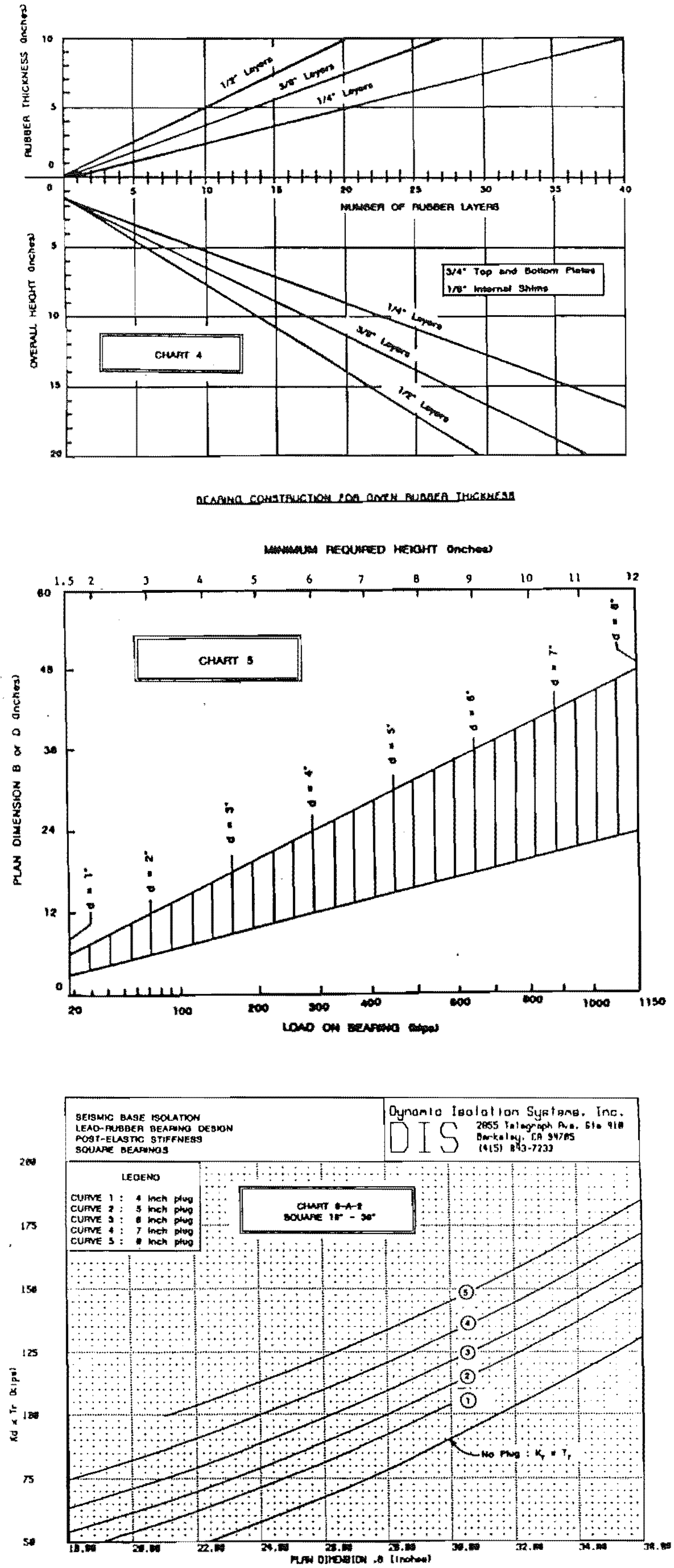


Fig. 2.2 ..... (continued)

displacement from step 3, obtain the maximum allowable load under earthquake loading. Compare this with the maximum dead load plus seismic live load plus earthquake induced axial load on the bearing. If the load capacity is less than applied load, the plan dimension must be increased or the rubber layer thickness decreased. Repeat from step 3 for the new average compressive stress.

5. Determine bearing construction.

From chart 4 determine the number of rubber layers and total height for the required rubber thickness.

6. Obtain lead plug diameter.

From chart 5 obtain the lead plug diameter required for the dead load on the bearing. Check that the plan dimension is greater than the minimum and less than the maximum. If not, decrease or increase the plan size, recalculate the compressive stress and return to step 3. Check that the overall height is greater than the minimum. If not, increase the rubber thickness and return to step 3.

7. Check overall bearing dimensions.

If the dimensions are satisfactory, check that any constraints imposed by the bearing layout are met. Adjust the bearing size if necessary.

8. Obtain post-elastic stiffness.

From chart series 6 obtain the value of k_d for each bearing for the selected plan dimension, lead plug size and rubber thickness.

9. Summarize final layout.

Summarize the final bearing size, construction and plug diameters for the final configuration.

It appears that this design procedure is aimed more for designing the lead-rubber bearings and their placements on site rather than for designing the whole structure. Hence, it was presented more as a design manual rather than providing the designer with a clear insight of the seismic behaviour of the structure or giving him/her a good feel of the sensitivity of the various parameters of base isolation system associated with the seismic response.

2.2.3 Andriono and Carr (1990) [A2,A3,A4]

This study was carried out to investigate in more detail the effects of various structural parameters and ground motion characteristics on the response of base isolated multistorey structures. The results were then used to develop two simplified analysis methods for practical design. The first proposed method which is called the Code-Type approach [A2] can be used to accurately estimate the inertia forces, not only at the level of the base isolation devices but throughout the entire height of the multistorey structure. This design procedure is suitable for a preliminary design or even a final design of uniform base isolated multistorey structures with an unisolated fundamental natural period, $T_{1(UI)}$, less than approximately 0.8 seconds. The second procedure, which is based on the Component Mode Synthesis method [A2], is suggested for final design purposes of base isolated multistorey structures with more irregular and/or more flexible superstructures.

The availability of these simple approximation methods means that inelastic time history analyses will no longer be necessary for practical design purposes. However, inelastic time history analyses may still be required to evaluate the inelastic behaviour of the superstructure under a very severe earthquake in order to ensure that the superstructure will have a satisfactory failure mechanism. The two design methods are briefly presented below.

a. Code-Type Approach

The proposed Code-Type approach is developed by adapting the well-known equivalent static lateral force analysis procedure to suit the seismic behaviour of base isolated multistorey structures. It was hoped that this similarity would help the designer to become familiar with this proposed approach. Fig. 2.3 shows a flow chart to illustrate the step-by-step procedure of this simple design method.

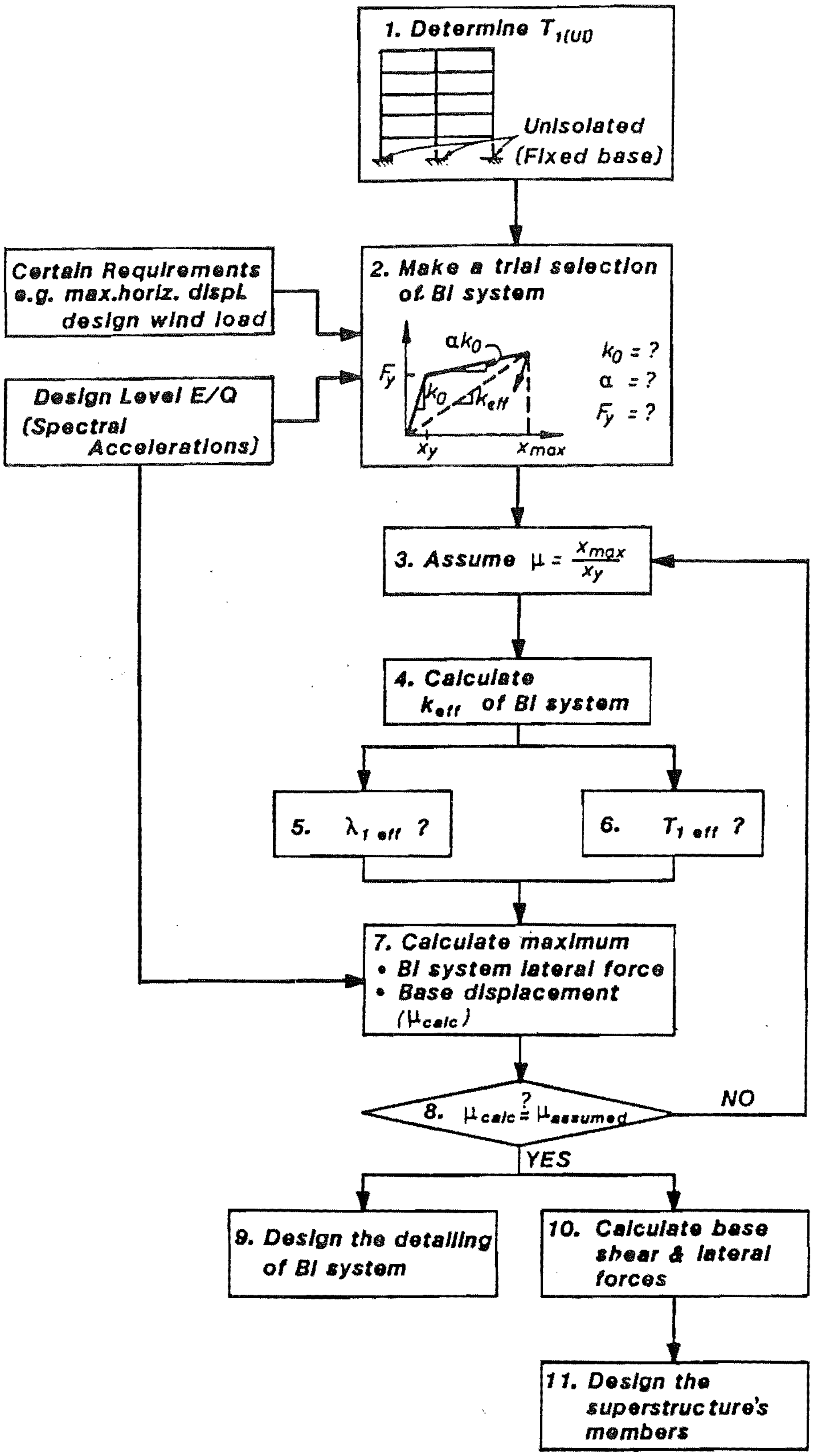


Fig. 2.3 Step-by-Step Design Procedure of the Code-Type Approach for Base Isolated Multistorey Structure

1. Determine the fundamental period of the unisolated superstructure $T_{1(UI)}$.

   This can be carried out as usual by assuming that the superstructure is not mounted on a base isolation system. At a preliminary design stage the approximate formulas as mentioned by some codes [C10,D4,R1,T1] can be used to estimate the fundamental period of this fixed base superstructure.

2. Make a trial selection of the base isolation system.

   The required reduction of lateral inertia forces is normally the main consideration for selecting or predicting the idealized bilinear hysteresis loop parameters of a base isolation system, i.e. its initial stiffness, k_o, its post-yield stiffness, αk_o, and its yield strength, F_y. Other requirements such as the maximum allowable horizontal displacements at working loads (due to wind and small earthquakes) and ultimate load levels (stability of the base isolation system) should also be considered. For this purpose a designer must know the design-level seismic load specified by the code for the particular site where the structure will be built, as well as the essential characteristics of a desirable base isolation system. The designer must therefore make an initial selection of k_o, α and F_y.

3. Assume the maximum base displacement under the design-level earthquake and calculate the so-called maximum displacement ductility ratio, $\mu_{assumed}$.

4. Obtain the effective (secant) stiffness of the base isolation system at the maximum base displacement by using Eq. 2.4 or from the chart shown in Fig. 2.4.

$$k_{eff} = k_o \left(\frac{1-\alpha}{\alpha} + \alpha\right) \qquad (2.4)$$

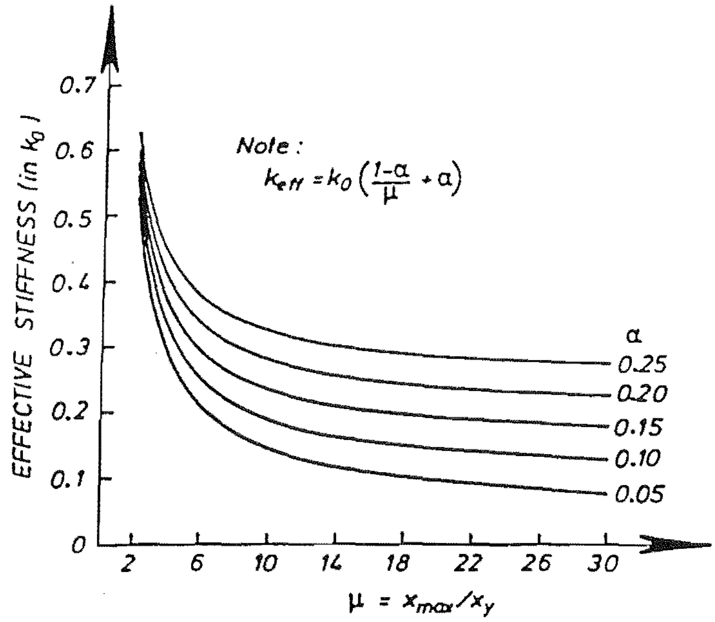


Fig. 2.4 Effective Stiffness of Base Isolation Systems with Bilinear Hysteresis Loop Model

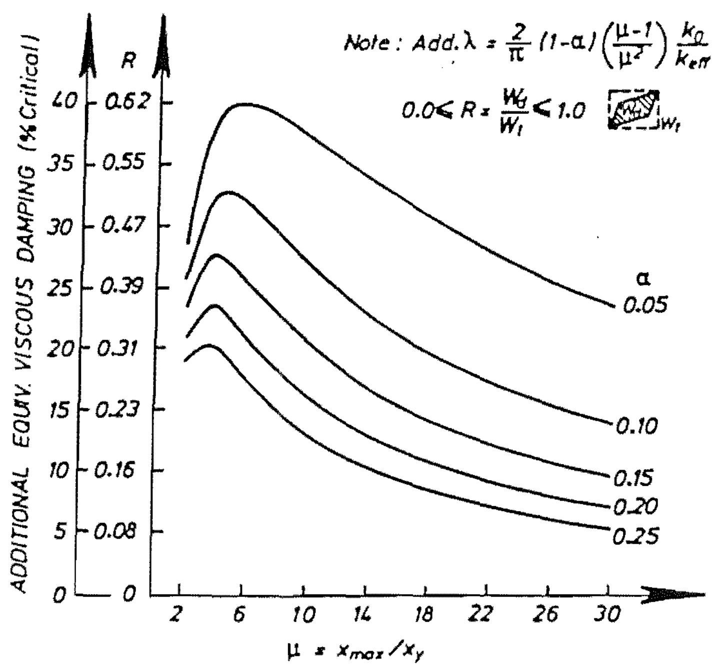


Fig. 2.5 Additional Damping of Base Isolation Systems with Bilinear Hysteresis Loop Model

5. Determine the increase in damping due to the hysteretic behaviour of the base isolation system using Eqs. 2.5 and 2.6 or from the chart shown in Fig. 2.5. Then calculate the effective damping of the structure as the sum of the inherent damping of the structure and this additional hysteretic damping.

$$\lambda_{add.} = E_h = \frac{2}{\pi} R \tag{2.5}$$

$$R = (1-\alpha)\left(\frac{\mu-1}{\mu^2}\right)\frac{k_0}{k_{eff}} \tag{2.6}$$

In this study R is called the hysteresis loop ratio, i.e. the ratio of the hysteresis loop area to the area of the circumscribing rectangle. This value will be used further in step 10.

6. Determine the effective fundamental period of the base isolated multistorey structure from the chart shown in Fig. 2.6. Note that this chart is developed for base isolated multistorey structures with uniform floor mass and interstorey stiffness. Charts for other variations of floor mass and interstorey stiffness may be developed later. In the absence of such charts a proper modal analysis should be conducted to calculate the effective fundamental period of the base isolated structure.

7. Based on the effective fundamental period and damping of the structure determine the maximum shear force of the base isolated structure from the appropriate acceleration spectra specified by the New Zealand loading code. Then calculate the maximum base displacement and the maximum displacement ductility ratio, μ_{calc}.

8. Compare the calculated maximum displacement ductility ratio, μ_{calc} with the maximum displacement ductility ratio, $\mu_{assumed}$ assumed in step 3.

If the difference between these two values is relatively great, say above 5% or so, steps 3 to 8 should be repeated. The calculated maximum

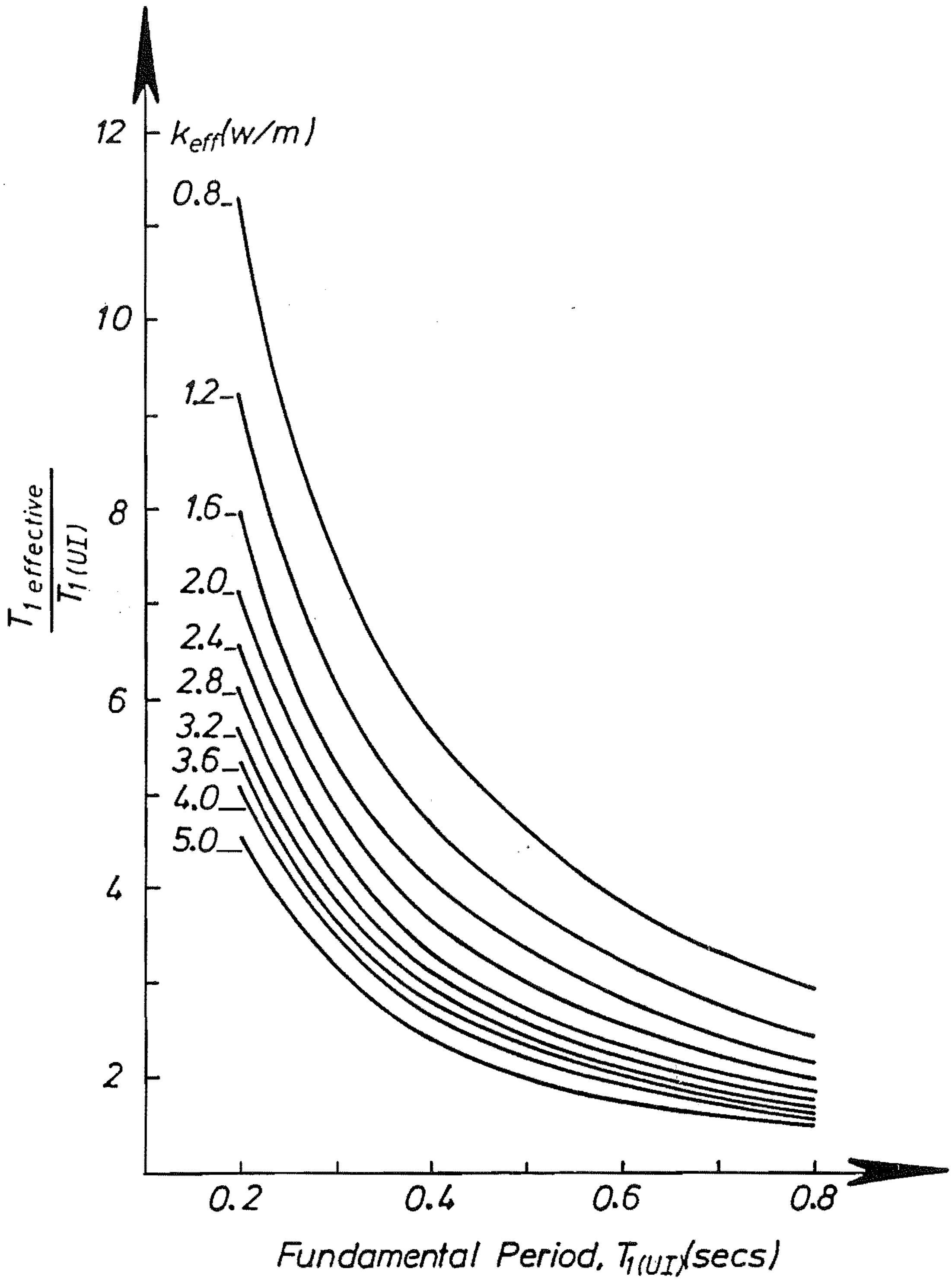


Fig. 2.6 Effective Fundamental Period of Uniform
Base Isolated Multistorey Structures

displacement ductility ratio may be used as a new assumed value until the two values converge. The convergence in the trial and error process is normally achieved very rapidly. Then the design process can be continued to step 9.

9. Design the details of the base isolation system.

Experimental data on tests of base isolation devices and design manuals for bearings can be used as a guidance to design the selected base isolation system in detail.

10. Determine the equivalent static lateral force distribution over the entire height of the multistorey structure.

The equivalent static lateral force, F_i at floor i can be accurately predicted by the following formula:

$$F_i = V \frac{W_i \, h_i^p}{\Sigma \, W_i \, h_i^p} \tag{2.7}$$

where V is the base shear, W_i and h_i are the weight and height of floor i respectively. The exponent p can be determined from the strong linear correlation with the hysteresis loop ratio of the base isolation system shown in Fig. 2.7. A modification factor, as shown in Fig. 2.8, should also be used to obtain the maximum value of the base shear of the superstructure from the shear force of base isolation system.

11. Design the members of the superstructure.

Once the lateral forces are satisfactorily determined the member forces in the structure can be computed and the members of the superstructure can be designed in more detail.

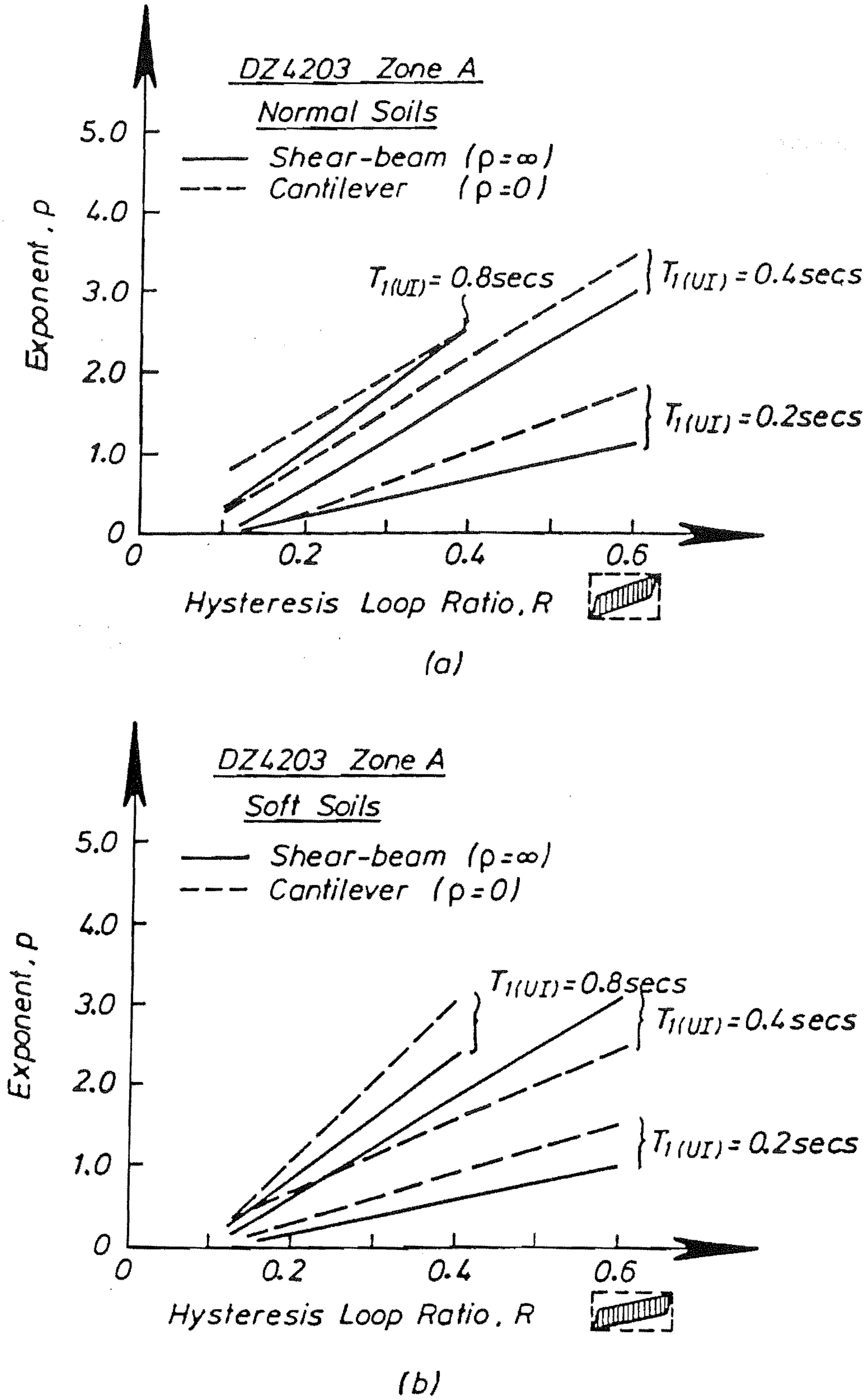


Fig. 2.7 Relationships between R and p for Different $T_{1\,(UI)}$ under New Zealand Design-Level Earthquakes for Zone A

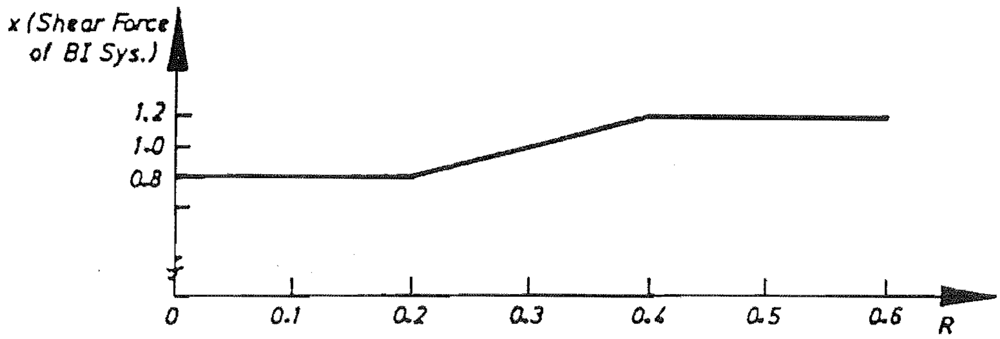


Fig. 2.8 Modification Factors Used for Predicting the Maximum Base Shear from the Base Isolation System's Maximum Shear Force

As in the use of the equivalent static lateral force procedure for nonisolated buildings, this Code-Type approach would be adequate for base isolated multistorey structures which have a uniform property configuration in all storeys or floors. The results of the investigation show that this simple approach can reliably predict the response of short to medium-rise base isolated structures with $T_{1(UI)}$ less than or equal to 0.8 seconds, with floor masses which do not differ by more than 25% in adjacent floors and where the lateral storey stiffness do not differ by more than 25% in adjacent storeys.

b. Component Mode Synthesis Method

In the design of unisolated structures, analysis methods using the modal superposition technique are normally employed if the equivalent static lateral force procedure is not able to satisfactorily predict the structural response. In a similar way the Component Mode Synthesis method was suggested as a means of analysis for more complex base isolated multistorey structures. In brief, this method treats a base isolated multistorey structure as two separate components, i.e. the superstructure and the base isolation system. The superstructure, which is expected to remain elastic under the design-level earthquake, can now be considered as a linear multi-degree-of-freedom system and the ordinary modal analysis procedure can employed independent of the inelastic behaviour of the base isolation system.

This scheme is still based on a step-by-step integration. However, since it is normally appropriate to approximate the structural response by incorporating the first few significant modes this method requires less computational effort when compared with the inelastic time history analysis. Further, the Component Mode Synthesis method gives the designer clearer insight into the structural response by showing modal contributions. However, it is desirable that this method should operate on the response spectrum analysis approach rather than the step-by-step integration.

2.2.4 Skinner, Robinson and McVerry (1993) [S6]

This work was a relatively extensive systematic study on the seismic responses of base isolated buildings where the parameters of the structure and isolation systems were varied over wide ranges. Both the seismic performance of linear structures on linear isolation systems and of linear structures on non-linear isolation systems were investigated. In the non-linear analysis, the bi-linear model was used to simulate the force-displacement relationship of the hysteretic non-linear base isolation systems. A numerical analysis method was used to investigate the effect of each parameter in the isolation systems on the seismic response of a base isolated building subjected to the N-S component of the 1940 El Centro earthquake.

It was found from this study that an early decision in the design of a seismically isolated structure is to determine whether a linear or nonlinear isolation system is required. The selection will be governed partly by the nature of the design criteria. As discussed in Ref. S6, nonlinear isolation systems can usually produce lower values of first-mode dominated response quantities, such as base shears and displacements, while linear systems are particularly effective at suppressing higher frequency responses. The two design procedures proposed for the design of isolated structures with linear and nonlinear isolators are briefly described below.

a. Design Procedure for Buildings with Linear Isolation Systems

Standard modal analysis procedures can be used to estimate the design responses of linear isolation systems. Initial estimates of displacements and base shears can be obtained from a simplified single-mass model because of the low participation factors of higher modes.

As a first approximation, the fundamental mode period, T_b, and damping factor, ζ_b, of a system with a high degree of linear isolation can be obtained by treating the structure as rigid, where

$$T_b = 2\pi \sqrt{\frac{M}{K_b}} \tag{2.8}$$

$$\zeta_b = \frac{C_b}{2\sqrt{M K_b}} \tag{2.9}$$

and the isolator has stiffness K_b and damping coefficient C_b, and supports a total mass M. The maximum base displacement X_b and base shear S_b can be computed from Eqs. 2.10 and 2.11.

$$X_b = S_D(T_b , \zeta_b) \tag{2.10}$$

$$S_b = M\ S_A(T_b , \zeta_b) \tag{2.11}$$

b. Design Procedure for Buildings with Bilinear Isolation Systems

Design criteria will usually involve acceptable base shears and displacements, and perhaps allowable shears at other levels of the structure and acceptable floor response spectra. The estimation of the seismic response for a structure with bilinear hysteretic isolation may proceed as below.

1. Select a trial isolation system.

   For design to a scaled El Centro type motion which gives base shear and base displacement as a function of Q_y / W, for various periods T_{b1} and T_{b2}, provide the possible combinations of parameters which produce responses meeting the design criteria. Here, Q_y / W is the ratio of yield force of the isolator to the weight of the structure. The period T_{b1} and T_{b2} relate to the elastic and post-yield stiffness K_{b1} and K_{b2} respectively.

2. Take a trial value of the base displacement X_b for the specified earthquake motion.

Calculate S_b, T_B and ζ_B from the hysteresis loop which is drawn for the chosen values of K_{b1}, K_{b2}, Q_y/W and X_b. For an assumed X_b, the bilinear loop gives the base shear S_b and ratio of base shear to the weight of the structure S_b/W as

$$S_b = Q_y\left(1 - \frac{K_{b2}}{K_{b1}}\right) + K_{b2}X_b \tag{2.12}$$

$$\frac{S_b}{W} = \frac{Q_y}{W}\left(1 - \frac{T_{b1}^2}{T_{b2}^2}\right) + \frac{4\pi^2 X_b}{g T_{b2}^2} \tag{2.13}$$

The effective stiffness is the secant stiffness

$$K_B = S_b / X_b \tag{2.14}$$

The equivalent linear period based on this stiffness is

$$T_B = T_{b2}\left[1 + \frac{g\,(T_{b2}^2 - T_{b1}^2)}{4\pi^2 X_b}\,\frac{Q_y}{W}\right]^{-\frac{1}{2}} \tag{2.15}$$

The equivalent viscous damping corresponding to the hysteretic damping is ζ_h, where

$$\zeta_h = \frac{2}{\pi}\,\frac{Q_y / W}{S_b / W}\left(1 - \frac{T_{b1}^2}{T_B^2}\right) \tag{2.16}$$

To obtain the total damping ζ_B, the viscous damping ζ_v must be added. ζ_v should be associated with a particular viscous damper coefficient C_b, which gives a fraction ζ_{b2} of critical viscous damping at period T_B. The corresponding fraction of critical viscous damping is $(T_B / T_{b2})\,\zeta_{b2}$. This definition gives

$$\zeta_B = \zeta_h + \frac{T_B}{T_{b2}}\,\zeta_{b2} \tag{2.17}$$

3. Use the earthquake displacement spectrum to find $S_D(T_B, \zeta_B)$, which is assumed to correspond to X_b, and estimate S_b from the hysteresis loop.

   This approximation assumes that the structural flexibility and damping has little effect on the first mode period and damping, as the structure is regarded as rigid to obtain T_B and ζ_B.

4. Check whether the base displacement and base shear of step 3 agree with the assumed displacement and corresponding base shear of step 2. If satisfactory convergence has not occurred, further iteration is required. New values of T_B and ζ_B can be calculated using the latest values of X_b and S_b.

5. Check the final estimates of X_b and S_b with the design criteria. If the values are not acceptable, or it is felt that improved values may be possible, select a new trial isolation system.

6. Check the higher mode responses.

   The elastic-phase isolation factor, $I(K_{b1})$, and non-linearity factor, $\underline{NL}$, are calculated as

$$I(K_{b1}) = T_{b1} / T_1(U) \qquad (2.18)$$

$$\underline{NL} = (\pi/2)\,\zeta_h \qquad (2.19)$$

   where $T_1(U)$ is the first mode period of the unisolated structure and ζ_h is as given in step 2. Using these parameters and the curves shown in Ref. S6 to estimate the ratios between the second and third-mode top-mass accelerations and the first-mode top-mass acceleration.

7. Repeat the calculations for any other required earthquake motions and perform response history analysis for a number of appropriate

accelerograms to confirm the results obtained with the spectra approach for the equivalent linear system.

Based on this study, for a building with proprietary selected linear isolation systems, the seismic response of the building is dominated by its fundamental mode. The second and higher modes make only a minor contribution to the response. Increasing viscous damping in the isolation system generally decreases the displacement response of the overall system, which is mainly governed by the first mode response, but increases the importance of the high frequency acceleration components. The earthquake attack on secondary systems of the structure may increase significantly with increasing isolator damping because of the enhanced high frequency response, although remaining less than in an unisolated structure.

2.2.5 Cui and Pan (1995) [C13,P1]

This research work was carried out to study the dynamic characteristics and seismic performance of frame buildings deforming in a shear-like manner mounted on linear and non-linear isolation systems. The deterministic and random responses of segmental buildings were also discussed in this study. The analysis process took into account the superstructure flexibility, base raft inertia as well as the interaction between the secondary system and the main structure. Parametric studies were carried out to investigate the influence of each parameter in the isolation system on the seismic response of both the base isolated buildings and their contents. In the nonlinear analysis, the bilinear hysteretic model was used to simulate the force-displacement relationship of the base isolation systems, and the statistical equivalent linearization method was used to estimate the root-mean-square response of the systems subject to random excitations. The following four topics were included in this study.

a. Dynamic Characteristics of Buildings with Linear Isolation Systems

Based on the base isolated models shown in Ref. C13, the combined effects of superstructure flexibility and base raft inertia on the dynamic characteristics were investigated. A series of parametric studies were carried out, and the effects of varying the stiffness and mass of the base isolation system on the frequencies and mode shapes were identified. Since the first mode response usually dominates the seismic response of a base isolated building, first-order solutions are obtained for the fundamental base isolated frequency and mode shape. For frame buildings deforming in a shear-like manner mounted on linear isolation systems, the second and higher modes have smaller participation factors when compared with the fundamental mode. The effectiveness of seismic isolation in reducing the higher mode response, by deflecting the ground motion energy through the orthogonality property, is clearly demonstrated.

b. Seismic Response of Buildings with Bilinear Isolation Systems

The root-mean-square (RMS) responses of frame buildings deforming in a shear-like manner mounted on bilinear seismic isolation systems subjected to random excitations were calculated by using the statistical equivalent linearization method. Parametric studies were carried out to investigate the effect of varying each parameter of the isolation system and superstructure on the seismic response of the base isolated buildings. The parameters considered were the initial and the post-yield isolated frequencies, the yield ratio is the ratio of the yield force to the total weight of a base isolated building, and the fundamental fixed base frequency of the superstructure.

The main purpose was to produce plots of the RMS response of a base isolated building mounted on a bilinear isolation system and excited by a random earthquake input as a function of the parameters of the bilinear isolation system and superstructure. From the studies, for a given set of system parameters, there is an optimal yield ratio which will minimize the base shear or base displacement for a given earthquake input. It indicated that the effect of varying superstructure flexibility on the base displacement and base shear was not significant. For frame buildings deforming in a shear-like manner supported on a bilinear isolation system, a single-degree-of-freedom model can produce a good approximation to the system.

c. Seismic Response of Secondary Systems

The response of secondary systems mounted inside a base isolated building is different from that of the systems directly mounted on the ground as the floor accelerations differ in severity and characteristics from the typical noise-like ground accelerations which generate them. The root-mean-square responses of secondary systems mounted in the base isolated buildings with linear and bilinear isolators subjected to a random excitation were calculated using the statistical equivalent linearization method. The effect of varying isolator and structural parameters on the response of the secondary system was investigated through parametric studies. The parameters considered in the analysis were the initial and post-yield isolator stiffness, the yield ratio of the base isolator, the damping coefficient of extra viscous dampers in the base isolator and the viscous damping of the superstructure and secondary systems.

Compared to the response in the fixed base buildings, the peak response of secondary systems are generally reduced by using base isolation systems. The frequencies of the first peaks of floor response spectra are close to the fundamental isolated frequency of the supporting system. For buildings with linear isolation systems, increasing the viscous damping of a secondary system can significantly reduce its response, when the natural frequency of the secondary system is close to the isolated frequency. Increasing the viscous damping in the isolation system would also have a similar effect on the response of secondary systems. For buildings with a bilinear isolation system, increasing the viscous damping of a secondary system significantly reduces the amplitude of floor response spectra. It noted that the viscous damping in the superstructure may help to reduce the response of the secondary systems.

d. Seismic Response of Segmental Buildings

This research work presented a preliminary study on the seismic response of an optimized segmental building. Both deterministic and random response analyses were carried out. In the deterministic response analysis, the N-S component of the 1940 El Centro earthquake ground motion is used as an earthquake input. In the random response analysis, the earthquake process is modelled as a filtered white noise. The segmental building can be regarded as a base isolated building, for which the superstructure is further divided into several segments interconnected by additional vibration isolators. In this study both the structure and the isolation systems were assumed to be linearly elastic.

The segmental building concept can be viewed as an extension of the conventional base isolation technique with a distributed flexibility in the superstructure. Absorption and dissipation of earthquake input energy are afforded by all vibration isolation systems in the segmental building, rather than by a single isolation system at the base level. From the investigations of this study, the segmental building possesses the ability to decouple the building from the harmful horizontal earthquake ground motions in a manner similar to that of the conventional base isolated building. While keeping the acceleration response low, a segmental building significantly reduces the base isolator displacement response compared with that of a base isolated building. The optimum parameters of the isolation systems depend on not only the dynamic properties of the building but also on the characteristics of the ground motion.

2.3 Design Codes

2.3.1 New Zealand National Society for Earthquake Engineering Recommendation (1979) [B3]

A study group constituted by the New Zealand National Society for Earthquake Engineering (NZNSEE) prepared this recommendation for the design and construction of base isolated structures. The philosophy of base isolation was reviewed, the tentative code provisions and design rules were recommended and the requirements for construction of base isolated structures and for maintenance of the devices were provided. The recommended code provisions to New Zealand Loading Code [C9] for buildings incorporating mechanical dissipating devices are presented below.

1. The following criteria shall be satisfied for the design of buildings incorporating flexible mountings and mechanical energy dissipating devices and where foundation rocking is not permitted.

2. The performance of the devices used is to be substantiated by tests. Proper studies are to be made towards the selection of suitable design earthquakes for the building with respect to site seismicity and geology. The proposed base isolated structure shall be analysed using a inelastic time history analysis.

3. The structural type factor S for base isolated structures shall be 0.7 relating to the period of the total system when the mechanical energy dissipators are yielding. The shear force carried by dissipators and bearings, V, so calculated, shall be used to determine the initial level of yielding of the energy mechanical dissipators.

4. Structural members protected by base isolation shall be sized using the results of the inelastic dynamic analysis at the design earthquake intensity.

5. The centre of the stiffness of the isolators in a horizontal plane shall be as close as possible to the centre of mass of the building in order to reduce the response resulting from torsional motion. The horizontal force at the level considered shall be applied at a design eccentricity, $e_b = 0.1b$, measured perpendicular to the loading where b is the maximum horizontal dimension of the building at the level, measured perpendicular to the direction of the loading.

6. The seismic force factor, C, for portions of base isolated buildings may be reduced compared to the values for unisolated buildings and design forces are obtainable from the results of the dynamic analysis. The interstorey deflections of the base isolated structure shall be obtained from the dynamic analysis for the design earthquake and shall be used for partition and glazing separations.

7. The minimum building separation to its neighbour's boundary shall include the maximum allowable lateral movement of the isolators together with 1.5 times the maximum interstorey drift obtained from dynamic analysis or 0.2% of the building's height, whichever is larger.

With regard to structural detailing, it was recommended that structures incorporating energy dissipators be detailed to deform in a controlled manner under an earthquake loading greater than that of the design earthquake. This may generally be achieved by provision of suitable margins of strength between ductile and non-ductile members and by attention to detailing, but without full capacity design procedures.

2.3.2 Structural Engineers Association of Northern California's Tentative Seismic Isolation Design Requirements (1986) [S10,S11]

These design requirements were developed specifically for designing seismic isolated buildings and to supplement the "Tentative Lateral Force Requirements" published by the Structural Engineers Association of California (SEAOC) in October 1985 [S10]. The approach and layout of the 1986 document, published by the Structural Engineers Association of Northern California (SEAONC), [S11] was chosen to parallel the 1985 document published by SEAOC as far as possible. Emphasis was placed on equivalent lateral force procedures, the level of seismic input was that required for the design of fixed-base structures a level of ground motion that has a 10% chance of being exceeded in a 50-year period as described by the recommended ground motion spectra of ATC-3-06 [A5].

The base isolator, including all connections and supporting structural elements, is required to be designed for the effects of full response at the level of approximately a 500-year return period ground motion. In these design requirements dynamic methods of analysis are permitted, and for some types of structures are required, but the simple static equivalent formulae provide a minimum level for the design. The design procedures can be briefly outlined as shown below.

a. Simple Static Equivalent Formula Procedure

Minimum earthquake displacements and forces on seismic isolated structures shall be based on the true deformation characteristics of the isolation system. An example set of force-deflection test curves used to determine maximum and minimum effective stiffness are shown in Fig. 2.9. The isolation system shall be designed and constructed to withstand minimum lateral seismic displacement, D, which acts in the direction of each of the main horizontal axes of the structure in accordance with the formula:

$$D = \frac{10\ ZNST}{B} \tag{2.20}$$

where Z is the seismic zone factor, N is the near-field coefficient proximity to active faults, S is the site coefficient based on the soil profile, T is natural period of the isolated structure as found from Eq. 2.21 and B is the damping coefficient which corresponds to the damping value in percentage of critical damping.

$$T = 2\pi \sqrt{\frac{W}{k_{min} g}} \tag{2.21}$$

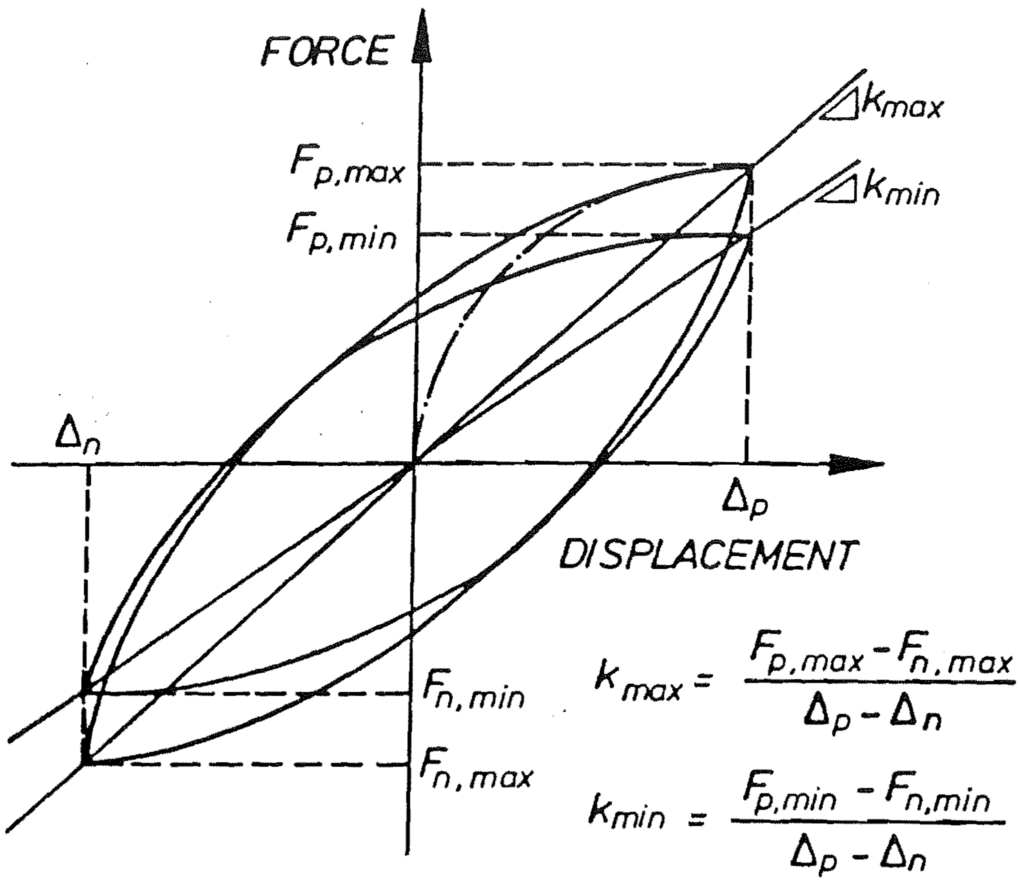


Fig. 2.9 Example of Force-Deflection Test Curves Used to Determine Maximum and Minimum Effective Stiffness

The total design displacement for the isolation system shall include additional displacement due to actual and accidental torsion calculated using the minimum effective stiffness of the isolation system. All structural components at or below the isolation interface shall be designed and constructed to withstand a minimum lateral seismic shear force, V_b, using all appropriate provisions for a non-isolated structure where

$$V_b = \frac{k_{max} D}{1.5} \tag{2.22}$$

and this equation gives peak seismic shear force on the above mentioned structural components without reduction for ductile response. The 1.5 factor is included to reduce the top shear to a level compatible with the allowable working stress specified in Ref. S10.

The elements of the superstructure above the isolation system shall be designed and constructed to withstand a minimum shear force V_s , using all the appropriate provisions corresponding to the R_w value for a non-isolated structure, where

$$V_s = \frac{2k_{max} D}{R_w} \tag{2.23}$$

and R_w is the ductility factor of the superstructure for conventional structures. Thus, for an isolated structure, the ductility factor is halved when compared with an unisolated structure; furthermore, R_w cannot exceed 8. In all cases, the value of V_s shall not be less than the following:

1. The lateral seismic force required by governing building codes for a fixed base structure with an empirical period equal to the isolated period.

2. The base shear corresponding to the design wind.

3. The yield level of the base isolation system.

This simple formula procedure may only be fully relied upon if the elastic, fixed base period of the building does not exceed 20% of its isolated period as determined from Eq. 2.21, otherwise a more rigorous analysis shall also be performed. Under the same requirement, the superstructure shall not have significant physical discontinuities in configuration or in the lateral force resisting system. Provided these requirements are satisfied the lateral inertia force distribution over the height of the structure is given

$$F_i = V_s \frac{W_i}{\Sigma W_i} \tag{2.24}$$

where W_i is a portion of the total weight W located at level i. This equation describes the vertical distribution of lateral force based on an assumed uniform distribution of seismic acceleration over the height of the superstructure. A similar assumption had been proposed by Skinner and McVerry [S4] as discussed in Section 2.2.1. The only difference lies on the specific limitation set for the two approaches. Skinner and McVerry applied the assumption for isolated structures with a fundamental period on fixed base not greater than 0.5 seconds, whereas SEAONC required that the fixed base fundamental period should not be greater than 20% of the effective period. It should be noted, however, that the same effective fundamental period can be obtained for short period structures mounted on a base isolation system with thin or fat hysteresis loops. As shown by Andriono and Carr [A2,A3,A4], the structures on a base isolation system with thin hysteresis loops have a uniform shear force distribution as predicted by Eq. 2.24, but if the structure has a fat loop base isolation system, this equation may lead to severely underestimated storey shears, especially in the upper storeys.

b. Dynamic Analysis Procedure

Dynamic analysis is required for buildings that are irregular or have an unisolated period greater than 20% of the isolated period. The analytical model shall be three-dimensional and shall include the deformational characteristics of the isolation system and superstructure. An analysis of lateral response shall be performed in both orthogonal directions of the building. If a response spectrum analysis is conducted, two separate analyses shall be performed, one using the maximum effective stiffness k_{max}, and the other using the minimum effective stiffness k_{min}, of the isolation system at the design displacement, unless the difference between the maximum and minimum effective stiffness is not more than 10%. In both cases the minimum effective damping value λ at the design displacement as estimated from Eq. 2.25 shall be used.

$$\lambda = \frac{1}{2\pi} \frac{\text{Area of hysteresis loop}}{k_{eff}\ \Delta^2_{max}} \qquad (2.25)$$

The results of these two analyses shall be considered acceptable if the calculated top displacement of the isolation system is within 10% of the design displacement used to determine the isolation properties. If a time history analysis is performed, at least three appropriate seismic inputs shall be used. The input time histories shall be selected from different recorded events and scaled such that their 5% damped response spectrum essentially envelopes the design spectrum with a margin not more than 10% lower at any period. Each analysis shall incorporate the minimum and maximum deformational characteristics of the isolation system, as mentioned earlier for the response spectrum analysis. The maximum response of these three analyses shall be used for design.

2.3.3 Uniform Building Code (1991) [I1]

The essentials of the Uniform Building Code (UBC) are very similar to the original SEAONC document [S11], but it has a number of differences in emphasis and has more restrictions on the use of the simple equivalent lateral force design procedure, requiring dynamic analysis under a wider range of circumstances. The major difference in the code is that it explicitly requires that the design must be based on two levels of seismic input as given below.

1. A design basis earthquake is defined as the level of earthquake ground shaking which has a 10% probability of being exceeded in a 50-year period. For this level of input the design provisions require the structure above the isolation system to remain essentially elastic.

2. The second level of input is defined as the maximum design earthquake, which is the maximum level of earthquake ground shaking that may be expected at the site within the known geological framework. This is taken as that earthquake ground motion that has a 10% probability of being exceeded in 250 years.

The philosophy of the code is that the isolator should be designed and tested for this level of seismic input, and all building separations and utilities that cross the isolation interface should be designed to accommodate the forces and displacements for this level of seismic input.

The procedures require the use of either static or dynamic analysis. The static procedure is limited to stiff low-rise buildings of regular configuration on stiff soil or rock sites away from active faults. The dynamic approach for all other situations can use linear response spectrum methods in certain cases, but in others may require time history analysis. If a linear dynamic analysis indicates that there may be some members stressed beyond allowable values, then a nonlinear time history analysis would be carried out and over-stressed elements could be found acceptable provided storey drifts calculated by the non-linear analysis satisfied the drift limits in the criterion. The design procedures can be briefly presented as follows:

a. Static Lateral Response Procedure

The static analysis under the requirements is based on the same formulae as in the SEAONC code with, a specific additional allowance for the effects of torsion. Thus, the minimum lateral seismic displacement, D, given by the formula Eq. 2.20 must be increased to the total design displacement, D_T, to allow for torsion; the minimum value of D_T is not less than 1.1D. The total maximum displacement, D_{TM} , required for verification of isolation system stability in the most critical direction of horizontal response shall be equal to 1.5 D_T.

The isolation system, the foundation and all structural elements at or below the isolation interface shall be designed and constructed to withstand a minimum lateral seismic force, V_b , using all of the appropriate provisions for an unisolated structure where

$$V_b = \frac{k_{max}\, D}{1.5} \qquad (2.26)$$

and this equation gives peak seismic shear force on the above mentioned structural components without reduction for ductile response. The 1.5 factor reflects the ratio of ultimate to working stress values.

The elements of the superstructure above the isolation system shall be designed and constructed to withstand a minimum shear force V_s, using all the appropriate provisions corresponding to the R_{wi} value for an unisolated structure, where

$$V_s = \frac{k_{max}\, D}{R_{wi}} \qquad (2.27)$$

and R_{wi}, force reduction coefficients, shall be based on the type of lateral force resisting system used for the structure above the isolation system shown on Ref. I1. The limits on a minimum shear force, V_s, are based on the same conditions as in the SEAONC code. The distribution of this lateral force over the height of the structure is taken to be uniform and same formula Eq. 2.24 used as in the SEAONC code. The maximum interstorey drift ratio of the structure above the isolation system shall not exceed $0.01/R_{wi}$.

b. Dynamic Lateral Response Procedure

The total maximum displacement, D_{TM} (equal to 1.5 D_T), is to be used for the verification of the stability of the isolator. When dynamic analysis is used, the static procedure must be followed since it provides lower bounds to the design quantities. The total design displacement of the isolator when dynamic analysis is used can not be less than 90% of D_T by the static formula, and the total maximum displacement, D_{TM}, not less than 80% of D_{TM} by the static formula. The design lateral shear force can be reduced to 80% of that for the static analysis and if time history analysis is carried out it can be reduced to 60% of the static force level.

The dynamic analysis technique can use response spectrum methods in some cases with an elastic response spectrum based on ATC-3-06 [A5]. If the structure is located on a soft soil site, or located within 15 km of an active fault, and has an isolated period of greater than 3.0 seconds, then a site-specific spectrum is required. A design spectrum must also be prepared for the maximum credible earthquake in order to determine the total maximum displacement for testing of the stability of the isolation system. This design spectrum is required to be at least 1.25 times the spectrum of the design basis spectrum. For time history analysis, pairs of horizontal ground motion records must be selected from at least three recorded events and are to be scaled to conform to the design basis earthquake, and if the site is within 15 km of an active fault, the time histories should incorporate near-fault phenomena.

When time history analysis is used as a design criterion, at least three pairs of horizontal input motions must be used and the maximum value of any design parameter over the three inputs used in the design. Additionally, the modelling of the isolation system must account for spatial distribution of the isolators and allow for torsion of the system. It should also be able to account for overturning and uplift on individual isolators. In both cases the effective stiffness k_{eff} and effective damping value λ of the isolation system are given by the following formula

$$k_{eff} = \frac{F^{+} - F^{-}}{\Delta^{+} - \Delta^{-}} \tag{2.28}$$

and

$$\lambda = \frac{1}{2\pi} \frac{\text{Total Area}}{k_{max} D^{2}} \tag{2.29}$$

where F^+ and F^- are the maximum positive and negative forces respectively; and Δ^+ and Δ^- are the corresponding maximum positive and negative test displacement; and k_{max} is the maximum effective stiffness of the isolation system at the design displacement D in the horizontal direction. The total area is taken as the sum of the area of the hysteresis loops of all isolator units.

2.4 Summary

The two important parameters, the effective fundamental period and the effective damping, have been considered for design of base isolated multistorey structures. Most of the methods used the effective fundamental period as a measure of the fundamental period shift due to the effect of the yielded base isolation system. Others used the effective damping which shows the increase in damping obtained as the result of the hysteretic behaviour of the base isolation device was utilised directly by some methods.

As mentioned above, a comparison of the design methods for multistorey structures showed that most of the suggested approaches were based on equivalent static analysis as commonly adopted by many loading codes. Some have taken into account the higher modes [C12,C13,P7,S6]. The form of the design aids vary from one method to another. Some methods required a code spectral acceleration, or elastic response spectra with a wide range of damping ratios, or inelastic response spectra, whereas others required the use of a series of design charts [D5,M2]. Different parameters have been considered for design purposes.

The three existing design codes have been reviewed in this chapter. NZNSEE requires the use of inelastic time history analysis as the only reliable design tool, whereas SEAONC [S11] permits the use of equivalent static force analyses with specific limitations. However, SEAONC requires three dimensional response spectrum or time history analyses based on the pseudo-elastic deformation characteristics of the base isolation system for the design of complex structures. The essentials of the UBC code [I1] are very similar to the SEAONC code, but it has a number of differences in emphasis and has more restrictions on the use of the simple equivalent lateral force design procedure, requiring dynamic analysis under a wider range of circumstances.

Although the SEAONC and the UBC documents are very similar in many ways, there is a fundamental difference between them. The first puts a premium on using static analysis and tends to force the designer toward regular superstructures with braced frames. The second, on the other hand, makes dynamic analysis necessary in more situations and creates an incentive to use dynamic analysis even where it is not mandatory in order to permit reductions in the design shear for the superstructure.

CHAPTER 3

THEORETICAL BACKGROUND OF DYNAMIC ANALYSIS AND STRUCTURE MODELLING

3.1 Introduction

The essential basis of seismic base isolation is to support a structure while providing a high degree of horizontal flexibility. This gives the overall structure a long effective period and low accelerations and inertia forces generated by earthquakes. The horizontal forces generated by typical design earthquakes are greatest on structures with low flexibility and low vibration damping. The seismic forces on such structures can be greatly reduced by supporting the structure on mounts which provide high horizontal flexibility and high vibration damping. For most structures, vertical seismic loads are relatively unimportant in comparison with horizontal seismic loads. Therefore, the main seismic attack on structures is the set of horizontal inertia force acting on the structural masses, these forces being generated as a result of horizontal ground acceleration.

The effects of an earthquake on a building depends on not only the properties of the earthquake ground motion but also on the dynamic characteristics of the building. For seismic isolation of buildings, it is necessary to include the flexibility of the structure itself and the interaction between the dynamics of the structure and the dynamics of the isolation system. Hence, dynamic analyses are required in order to be able to predict the response of structures subjected to dynamic loading. In these analysis methods, the real structures are represented by appropriate analytical models which can be described mathematically.

The complexity of an analytical model is determined by the real structural properties and behaviour it must represent. For most building structures a lumped mass model, usually with the whole storey mass lumped at the floor level, is generally all that is required. Therefore, the lumped-mass model is used in this study because the mass of the system is assumed to be represented by a finite number of point masses.

3.2 Equations of Motion

If the dynamic loading is a known function of time, then a deterministic method for solving the questions can be applied. The equation of motion for a multi-degree of freedom structure subjected to a time varying load can be written as below :

$$[M]\{\ddot{v}\} + [C]\{\dot{v}\} + [K]\{v\} = \{P(t)\} \qquad (3.1)$$

where [M], [C] and [K] are the mass, damping and stiffness matrices, $\{\ddot{v}\}$, $\{\dot{v}\}$ and $\{v\}$ are the accelerations, velocities and displacements of the structure and $\{P(t)\}$ is the time varying loading on the structure.

For a multistorey building, it is reasonable to lump the mass of the structure at certain nodes at which the translational degrees of freedom are defined. In this case, the lumped-mass matrix has a simple diagonal form. The off-diagonal terms of this matrix vanish since an acceleration of any mass point produces an inertia force at that point only [C8]. It is also usually assumed, in most building analyses, that the mass remains constant with time. The stiffness of the structure has a major effect on the design of a multistorey building. The stiffer the structure the shorter the natural periods of free vibration and the smaller the displacements under the earthquake excitation. The possible change of the structural stiffness due to inelastic or non-linear actions within the structural components should be calculated during the dynamic analysis. The direct stiffness method [R3] can be used to assemble the stiffness matrix of individual members into the global stiffness matrix of the structure.

It must be noted that the use of linear viscous damping is a mathematical convenience. The assumption of linear viscous damping which provides the simplest mathematical model of damping that is directly proportional to the velocity [C11]. If a deterministic non-linear time history analysis is used, the damping matrix is usually assumed to be a combination of the mass and stiffness matrices, thus

$$[C] = \alpha[M] + \beta[K] \qquad (3.2)$$

where the coefficient α and β are specified or computed by specifying the fraction of critical damping at two mode numbers.

In Eq. 3.1, the displacements are the total displacements of the structure measured from some fixed reference point. Usually most engineering calculations use relative displacement, $\{u\}$, i.e. displacement of the structure with respect to its foundation. As illustrated in Fig. 3.1, the total displacement, $\{v\}$, can be represented by a combination of this relative displacement, $\{u\}$, and the ground displacement, $u_g(t)$.

$$\{v\} = \{u\} + \{r\}u_g(t) \tag{3.3}$$

where $\{r\}$ is the displacement of the structure due to a unit ground displacement, then substituting Eq. 3.3 into Eq. 3.1 leads to

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=\{P(t)\}-[M]\{r\}\ddot{u}_g(t)-[C]\{r\}\dot{u}_g(t)-[K]\{r\}u_g(t) \tag{3.4}$$

Given the ground acceleration $\ddot{u}_g(t)$, this will have to be integrated with respect to time to give $\dot{u}_g(t)$ and $u_g(t)$. If the ground displacement history $u_g(t)$ is known it will have to be differentiated with respect to time to give the velocity and acceleration histories of the ground. If the ground motion is considered to be uniform over the site, i.e. travelling wave effects are not considered and the foundation is considered to undergo a rigid-body displacement, then Eq. 3.4 may be considerably simplified.

For a rigid-body displacement, no forces are generated within the structure. This is a necessary property of any member or structure stiffness matrix.

$$[K]\{r\} = \{0\} \tag{3.5}$$

If the usual assumption that damping forces are considered to be only due to relative velocities is made then

$$[C]\{r\} = \{0\} \tag{3.6}$$

and Eq. 3.4 becomes

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{P(t)\} - [M]\{r\}\ddot{u}_g(t) \tag{3.7}$$

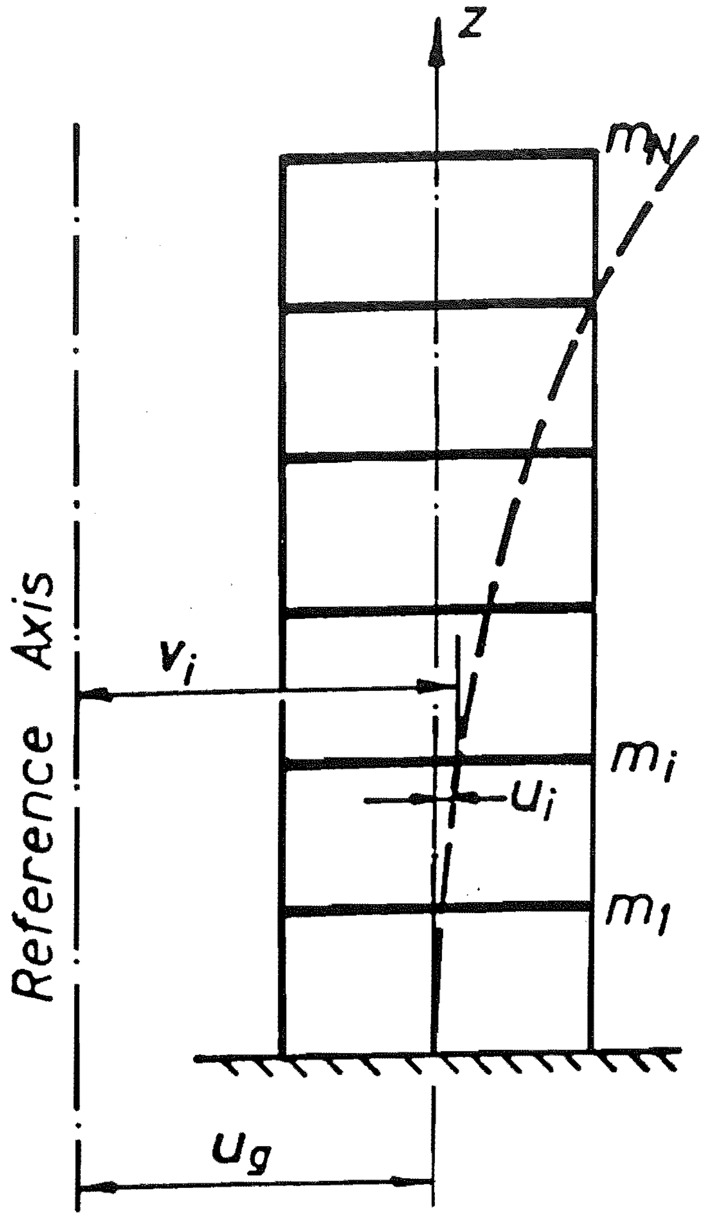


Fig. 3.1 Relative and Total Motion of a Multistorey Structure with Rigid Translation

If $\{P(t)\}$ is equal to $\{0\}$, then the term $-[M]\{r\}\ddot{u}_g(t)$ can be treated as the effective earthquake load. The negative sign has little significance in earthquake response analysis and often is ignored. However, it is important if total displacements or accelerations are required such as in the computation of floor acceleration spectra.

3.3 Modal Analysis

The mode shape and natural frequency functions of the base isolated structure can be derived from the undamped free vibration equations and boundary conditions. Taking the equation of undamped free vibration, where no loads are assumed to act upon the structure, the N degree-of-freedom equation of equilibrium becomes

$$[M]\{\ddot{u}\} + [K]\{u\} = \{0\} \tag{3.8}$$

and assuming simple harmonic motion $\{u\}_i = \{\phi\}_i\, Y_i \sin \omega_i\, t$ then the equation of free-vibration simplifies to

$$-\omega_i^2[M]\{\phi\}_i + [K]\{\phi\}_i = \{0\} \qquad i = 1, 2, ..., N \tag{3.9}$$

where $\{\phi\}_i$ and ω_i are the ith mode and frequency of free vibration respectively.

The N vectors $\{\phi\}_i$ form a basis set of vectors in that any vector in the N dimensional space may be represented as a combination of the mode shapes

$$\{u(t)\} = [\phi]\{Y(t)\} \tag{3.10}$$

where $\{Y\}$ are the modal amplitudes and $[\phi]$ is the modal matrix in which each column is a mode shape. Substituting Eq. 3.10 into Eq. 3.7 and pre-multiplying by $[\phi]^T$ gives

$$[M^*]\{\ddot{Y}\} + [C^*]\{\dot{Y}\} + [K^*]\{Y\} = -\{L^*\}\ddot{u}_g(t) \tag{3.11}$$

where $[\phi]^T[M][\phi] = [M^*]$, $[\phi]^T[C][\phi] = [C^*]$, $[\phi]^T[K][\phi] = [K^*]$ and $[\phi]^T[M]\{r\} = \{L^*\}$.

From the properties of orthogonality of the mode shapes it can be shown that $[M^*]$, $[C^*]$ and $[K^*]$ are diagonal matrices with

$$M_i^* = \{\phi\}_i^T [M] \{\phi\}_i \ , \qquad C_i^* = \{\phi\}_i^T [C] \{\phi\}_i = 2\lambda_i \, \omega_i \, M_i^* \ ,$$

$$K_i^* = \{\phi\}_i^T [K] \{\phi\}_i = \omega_i^{\,2} M_i^* \quad \text{and} \quad L_i^* = \{\phi\}_i^T [M] \{r\}.$$

where λ_i is the fraction of critical damping in the ith mode of free vibration. Thus for each mode Eq. 3.11 becomes

$$M_i^* \, \ddot{Y}_i + C_i^* \, \dot{Y}_i + K_i^* \, Y_i = -L_i^* \, \ddot{u}_g(t) \qquad i = 1, 2, ..., N \tag{3.12}$$

and then dividing though by M_i^*, the Eq. 3.12 can be written as

$$\ddot{Y}_i + 2\lambda_i \omega_i \dot{Y}_i + \omega_i^2 Y_i = \frac{-L_i^*}{M_i^*} \, \ddot{u}_g(t) = \frac{\{\phi\}_i^T [M] \{r\}}{\{\phi\}_i^T [M] \{\phi\}_i} \, \ddot{u}_g(t) = -PF_i \, \ddot{u}_g(t) \tag{3.13}$$

where the term PF_i is called the participation factor for the ith mode and it is an indicator of how much the ith mode is excited by the ground acceleration in the direction of the earthquake and indicates the importance of the contribution of the ith mode to the displacements of the structure.

The response spectrum analysis is an alternative method of analysis. In this method, the maximum displacement and force responses for each mode of the structure can be obtained directly, by reference to appropriate earthquake response spectra as a function of the modal natural periods and damping ratios, instead of evaluating them at each time step during the earthquake history. However, in order to determine the total maximum response, it is unreasonable to just add these maximum modal responses because these maxima may not occur at the same time. It is almost impossible to combine these modal responses to obtain the maximum response of the multi-degree of freedom system. The problem is usually resolved by relying on a statistical combination of the modal responses. The simplest and most popular method is the SRSS combination method which determines the total maximum responses as the Square Root of the Sum of the Squares of the modal responses considered.

A ductile structure, at the design level of response, is no longer linearly elastic and thus the principle of superposition is no longer valid and modal analysis methods are not applicable. The inelastic methods of dynamic analysis are usually based on deterministic time-history analyses of the structure subjected to known ground motion. To deal with non-linearity, a step-by-step integration technique of solving the equations of motion is required for the dynamic analysis. In this method, an actual or simulated time dependent earthquake accelerogram is applied to the base and the corresponding response-history of the structure during the applied motion can be computed step-by-step by taking into account any changes of the structural properties at each prescribed time interval.

3.4 Structure Modelling

The analytical models used in this study to represent a wide variety of base isolated multistorey structures are presented. The description includes the consideration of the soil-foundation system and base isolation system as well as that of the superstructure. The Rayleigh damping model used includes the effects of foundation damping within the damping model.

3.4.1 Soil-Foundation System

Most building footings are square or rectangular and are partially embedded foundations. During dynamic loading the soil immediately below the footing foundation will undergo higher shear strains than does the soil remote from the footing base [P6]. This condition can be represented as a fictitious two layered system with an upper and lower layer. The soil shear modulus G in the upper layer may degrade because of the higher soil shear strains occurring in this layer. The lower layer is assumed to possess its initial shear modulus G_{max} because the soil shear strain is relatively small. This soil-foundation model will lead to a rectangular partially embedded foundation resting on a fictitious layer over an elastic half-space as shown in Fig. 3.2.

A simple element cannot be used to represent the actual stress-strain behaviour of soil materials. Based on combinations of single mechanical element to form a composite model, the Voigt-Kelvin model [L3,V1] is considered as the simplest composite mechanical model. The Voigt-Kelvin model was represented by a parallel combination of a single linear spring and a single dashpot with the requirement that the principle of equal strain between two elements must

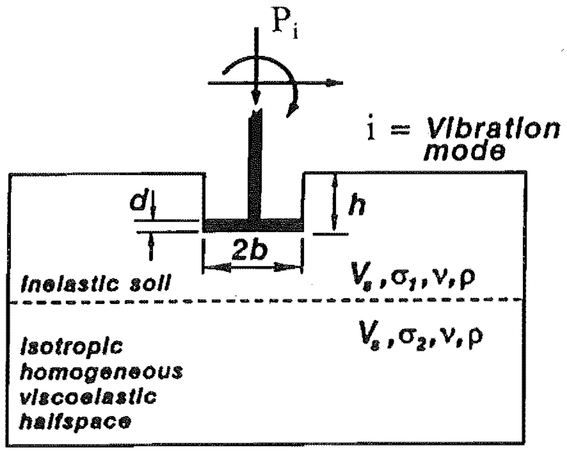


Fig. 3.2 Fictitious Layer over an Elastic Half-Space

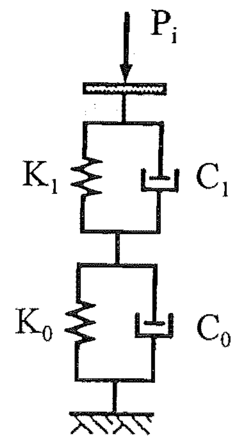


Fig. 3.3 Soil Mechanical Model

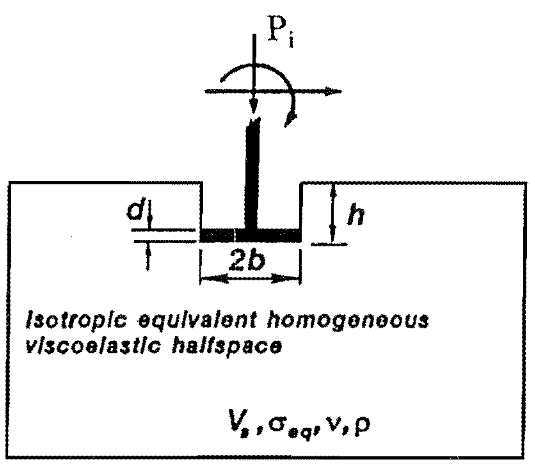


Fig. 3.4 Equivalent Elastic Half-Space

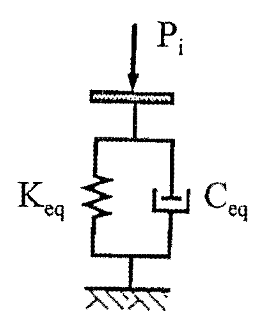


Fig. 3.5 Equivalent Soil Mechanical Model

be satisfied. By considering the use of the soil model as described above, the proposed soil-foundation mechanical model is shown in Fig. 3.3. In this study, the soil-foundation mechanical model used consists of a series of two Voigt-Kelvin models. One Voigt-Kelvin model is associated with the behaviour of the fictitious upper layer and the second one represents the behaviour of the lower soil layer.

As shown in Fig. 3.3, P_i is associated with the applied load relating to the ith displacement mode. The term K_1 and K_0 are the upper and lower soil-foundation stiffness associated with the apparent shear modulus G_1 and the maximum soil shear modulus $G_0 = G_{max}$ respectively. C_1 and C_0 are the damping coefficients for the upper and lower layers respectively. In this case, the damping coefficients include the radiation damping and material damping of the corresponding soil layer. The effects of soil material damping on the soil-foundation stiffness and damping coefficients will be discussed in Section 3.5.3.

The soil-foundation impedances can be computed by transforming the proposed soil-mechanical model shown in Fig. 3.3 to those shown in Fig. 3.4 or 3.5. The soil-foundation impedance will be discussed in Section 3.5. Terms K_{eq} and C_{eq} in Fig. 3.5 are the equivalent soil-foundation stiffness and damping coefficients respectively. To be able to transform to the equivalent soil-mechanical model, the force equilibrium between two Voigt-Kelvin models in series shown in Fig. 3.3 must be satisfied. This leads to

$$P_1 = P = K_1 \, y_1 + C_1 \frac{dy_1}{dt} \tag{3.14}$$

$$P_0 = P = K_0 \, y_0 + C_0 \frac{dy_0}{dt} \tag{3.15}$$

where K_1, K_0, C_1 and C_0 are the stiffness and damping coefficients for the upper and lower layers respectively and y_1, y_0 are the displacements of the upper and lower layers respectively.

Taking the first derivative of Eqs. 3.14 and 3.15 with respect to time then each of the derivative equations of displacement with respect to time becomes

$$\frac{dy_1}{dt} = \frac{1}{K_1}\frac{dP}{dt} - \frac{C_1}{K_1}\frac{d^2y_1}{dt^2} \tag{3.16}$$

$$\frac{dy_0}{dt} = \frac{1}{K_0}\frac{dP}{dt} - \frac{C_0}{K_0}\frac{d^2y_0}{dt^2} \tag{3.17}$$

and the total displacement of the system is the summation of the displacement of each element yielding

$$y = y_1 + y_0 \tag{3.18}$$

Then taking the first derivative of Eq. 3.18 with respect to time becomes

$$\frac{dy}{dt} = \frac{dy_1}{dt} + \frac{dy_0}{dt} \tag{3.19}$$

Substituting Eqs. 3.16 and 3.17 into Eq. 3.19 and rearranging in terms of the global displacement y leads to

$$\frac{dP}{dt} = \frac{K_1K_0}{K_1 + K_0}\frac{dy}{dt} + \frac{C_1K_0 + C_0K_1}{K_1 + K_0}\frac{d^2y}{dt^2} \tag{3.20}$$

Integrating Eq. 3.20 gives

$$P = K_{eq}\,y + C_{eq}\frac{dy}{dt} \tag{3.21}$$

where K_{eq} and C_{eq} are the equivalent stiffness and damping coefficients of the system as shown in Fig. 3.5 from which

$$K_{eq} = \frac{K_1K_0}{K_1 + K_0} \tag{3.22}$$

$$C_{eq} = \frac{C_1K_0 + C_0K_1}{K_1 + K_0} \tag{3.23}$$

Given K_1, K_0, C_1 and C_0, the values of equivalent stiffness and damping coefficients K_{eq} and C_{eq} of the system can then be computed.

3.4.2 Base Isolation System

The successful seismic isolation of a particular structure depends on the appropriate choice of the base isolation devices. The basic features of an isolation system are identified as:

1. An increased flexibility so that the natural period of the structure is increased sufficiently to shift the frequency of the structure out of the range of dominant frequency of earthquake.

2. A capacity for dissipating earthquake energy for resisting excessive horizontal displacement at the base of the building.

It is also necessary to provide an adequate seismic gap (between the structure and the surrounding foundation) which can accommodate the isolator displacements.

Many different forms of practical base isolation systems have been developed to provide seismic protection for buildings, including laminated elastomeric rubber bearings, lead rubber bearings, yielding steel devices, friction devices (PTFE sliding bearings), lead extrusion devices and combination of sleeved piles and mild steel energy dissipators [B5].

For the purpose of controlling the base displacement and resisting the wind load or small base disturbances, most of the seismic base isolation systems are designed to have non-linear hysteretic characteristics. All of these systems can be categorized as displacement amplitude dependent devices. Their hysteretic behaviour is a function of the deformation imposed on the system as described in Refs. A2,B6,S5 and S6.

Most of the practical isolation systems involve isolators with non-linear hysteretic characteristics. Many kinds of analytical models have been developed to simulate the non-linear behaviour of various isolation systems such as the bilinear model, trilinear model, Ramberg-Osgood model, etc.. The yielding devices can generally be represented by a bilinear hysteresis model with the bilinear factor from 0% of the initial stiffness in the yielding steel devices to approximately 15% in the lead rubber bearings. Therefore, bilinear and elasto-plastic hysteretic models shown in Figs. 3.6 and 3.7 are used in this study to represent these various bearings; the

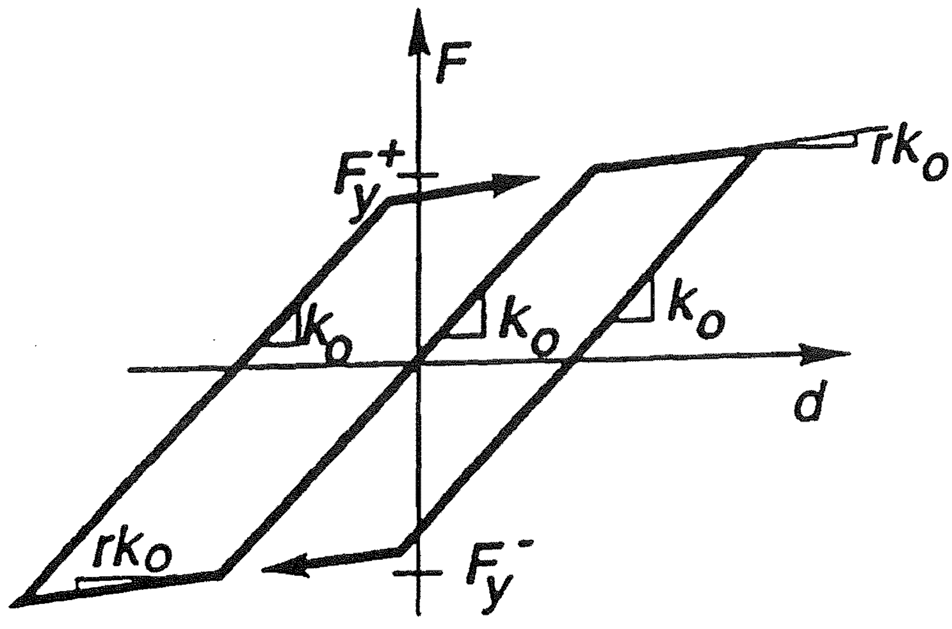


Fig. 3.6 Bilinear Model

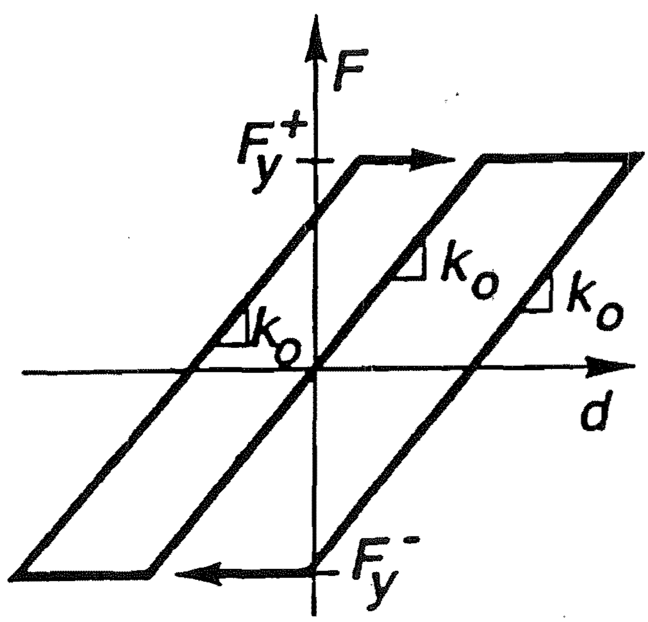


Fig. 3.7 Elasto-Plastic Model

bilinear hysteretic model for lead rubber bearings, and the elasto-plastic hysteretic model for yielding steel devices, friction devices (PTFE sliding bearings), and lead extrusion bearings.

3.4.3 Superstructure

In the case of a base isolated structure, its earthquake response can be approximated by a fundamental mode response in which the structure moves as a rigid body mounted on the isolation system. Many analyses normally use a frame structure deforming in a shear-like manner to simulate the behaviour of the superstructure in the seismic analysis of base isolated structures. In this frame structure, the floor slabs and beams are assumed to act as infinitely stiff members so that the lateral deflections result only from the column flexure without rotation at the joints. This frame structure deforming in a shear-like manner starts by investigating the seismic response of base isolated multistorey structures, as used earlier by Lee and Medland [L1,L2].

One advantage of a base isolated building is the ability to ensure that the superstructure behaves essentially linearly elastically during the design level earthquake. Fig. 3.8 shows the uniform superstructure model used in this study. In this model, the superstructures are designed as uniform mass and stiffness over their height. For simplicity a one dimensional flexural member with lumped mass at each floor was adopted to model the frame structure deforming in a shear-like manner while only one horizontal displacement at each floor was allowed.

Plastic deformation in the superstructure may occur if an earthquake of magnitude greater than the design level earthquake occurs. The effects of these plastic hinges on the seismic performance of a base isolated building were also studied using a moment-resistant frame model. In this model, all joints have three degrees of freedom with a lumped mass representation. The horizontal degrees of freedom of the nodes at the same floor are coupled to each other so that all horizontal displacements at the floor level are the same. Based on the capacity design method [P2,P5], plastic hinges were restricted to the beam ends and the ground floor column bases.

A Rayleigh damping model, where the damping is proportional to the mass and stiffness matrices of the structure, is used with 5% of critical damping being assumed to occur in the first and tenth modes for the structures. This is to ensure that the higher modes of the structure are

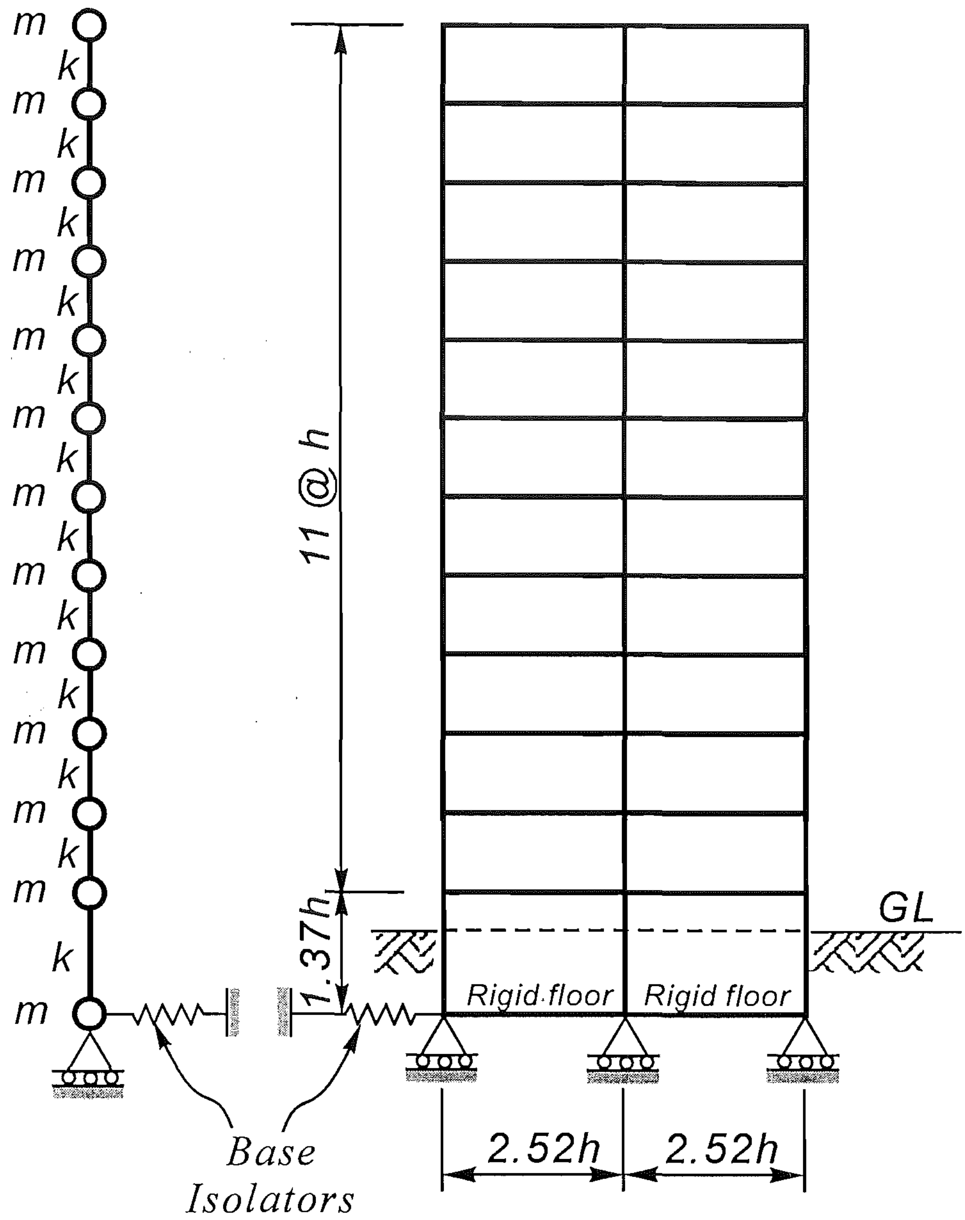


Fig. 3.8 Uniform Twelve-Storey Superstructure Model

sub-critically damped [C1] and that the lower modes have, in an average sense, the expected 5% of viscous damping.

3.5 Soil-Footing Foundation Response and Impedance

The actual soil beneath the foundation may be physically complex and no exact mathematical solution seems possible for the general problem of soil-foundation interaction [G1]. The soil-foundation stiffness and damping coefficients become more important when the lumped parameter method is used during analyses. Attempts to prove the validity of the lumped parameter model have been carried out either by analytical methods [L4] or by laboratory tests [R2]. In this study the lumped parameter method was used for the analysis for the following reasons; firstly, this method is relatively simple, requires less computational effort; secondly, this method can easily be incorporated in a general purpose structural dynamic program working in the time domain; thirdly, the calculated foundation impedance can be used in the interaction response of a variety of building structures.

The dynamic responses of foundation vibrations have been investigated for the embedded foundations [J3,N2,W2]. Results from the analytical investigations and laboratory tests, however, indicate that under dynamic loading both the stiffness and damping coefficients are frequency dependent. In reality, the footing foundations are not necessarily fully embedded but both arbitrarily shaped and partially embedded. The dynamic responses of arbitrarily shaped and partially embedded foundations for all vibration modes were reported by Gazetas et al. (1985) and Gazetas and Tassoulas (1987). As this is a two-dimensional model only the vertical and horizontal vibration modes are to be considered in this study. It was found during laboratory tests that the degree of foundation embedment has a significant effect on the foundation responses. Therefore, the responses of arbitrarily shaped and partially embedded foundations are used in this study.

In this model, the base isolated and segmental structures with foundation compliance include a rocking mode in the structure due to the vertical compliance. This will, in general, increase the displacements and decrease the forces on the building.

3.5.1 Vertical Stiffness and Damping Coefficients

a. Vertical Static Stiffness Coefficient

The method used is an extension of the dynamic response of the surface foundation resting on the elastic half-space was presented by Dobry and Gazetas (1986) [D1]. As shown in Fig. 3.9, the vertical static stiffness of an arbitrary shaped surface foundation, $K_{sv,sur}$ resting on elastic half-space can be written as

$$K_{sv,sur} = \frac{2\,G\,L}{1 - \nu}\,S_z \tag{3.24}$$

where G, L and ν are the soil shear modulus, the one-half of foundation length and the soil Poisson's ratio respectively. The vertical static stiffness parameter, S_z can be obtained by

$$S_z = 0.8 \qquad \text{for} \quad \frac{A_b}{4\,L^2} < 0.02 \tag{3.25}$$

$$S_z = 0.73 + 1.54\left(\frac{A_b}{4\,L^2}\right)^{0.75} \qquad \text{for} \quad \frac{A_b}{4\,L^2} > 0.02 \tag{3.26}$$

The effect of the trench foundation on the vertical stiffness leads to the trench coefficient which is denoted by I_{tre} which can be written as

$$I_{tre} = 1 + \frac{1}{21}\,\frac{D}{B}\left\{1 + \frac{4}{3}\,\frac{A_b}{4\,L^2}\right\} \tag{3.27}$$

where D is the depth of embedment, B is one-half of foundation width where L > B and A_b is the area of the foundation base.

Another aspect of the embedded foundation response is the sidewall effect. When the vertical sidewall is in contact with the surrounding soil, part of the applied load is transmitted to the ground through shear stresses acting along the vertical sidewall. This effect leads to the sidewall coefficient which is denoted by I_{wall} which can also be written as

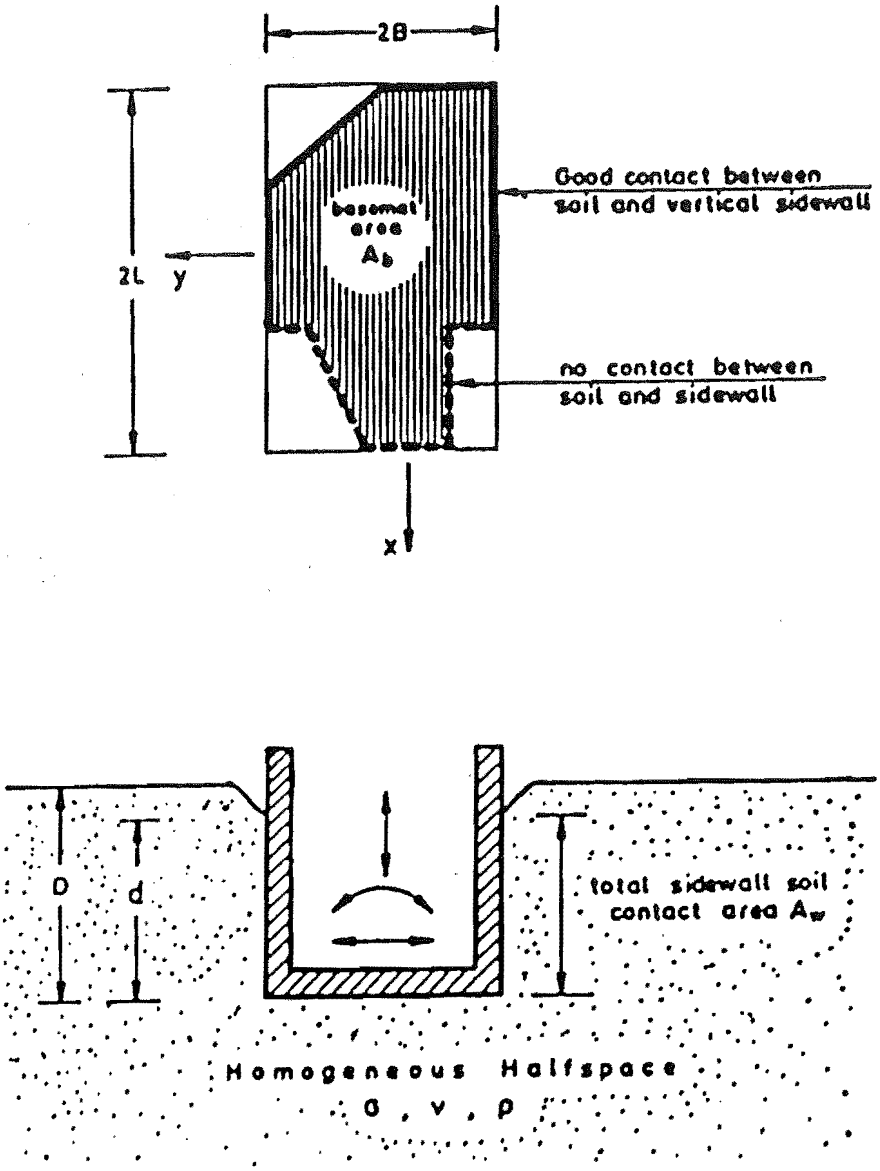


Fig. 3.9 Arbitrary Shape Embedded
Footing on Half-Space

$$I_{wall} = 1 + 0.19 \left\{\frac{A_s}{A_b}\right\}^{\frac{2}{3}} \tag{3.28}$$

where A_s is the area of the sidewall-soil contact. The static vertical stiffness of the partially embedded foundation, $K_{sv,emb}$ can be calculated by considering all of the coefficients and giving

$$K_{sv,emb} = K_{sv,sur}\ I_{tre}\ I_{wall} \tag{3.29}$$

b. Vertical Dynamic Stiffness Coefficient

Taking into account the effect of excitation frequency on the fully embedded foundation leads to the dynamic stiffness coefficient denoted by $k_{v,emb}$. The vertical dynamic stiffness of a fully embedded foundation, $K_{dv,emb}$ can be computed from Eq. 3.29 and becomes

$$K_{dv,emb} = K_{sv,emb}\ k_{v,emb} \tag{3.30}$$

and the dynamic stiffness coefficient of surface foundation, $k_{v,sur}$ against the dimensionless frequency a_0 for any values of L/B can be determined [G1]. The relation of $k_{v,emb} / k_{v,sur}$ against the dimensionless frequency for any value of D/B can be obtained by

$$k_{v,emb} = \{1 - 0.09 \left(\frac{D}{B}\right)^{0.75} a_0^2\}\ k_{v,sur} \tag{3.31}$$

and for the foundation placed in a trench without sidewall, the dynamic stiffness coefficient, $k_{v,tre}$ can be expressed by

$$k_{v,tre} = \{1 + 0.09 \left(\frac{D}{B}\right)^{0.75} a_0^2\}\ k_{v,sur} \tag{3.32}$$

In the case of an partially embedded foundation, the corresponding dynamic stiffness coefficient can then be computed by interpolating between Eqs. 3.31 and 3.32, and then substituting the calculated dynamic stiffness coefficient into Eq. 3.30 to obtain the vertical dynamic stiffness of a partially embedded foundation.

c. Vertical Damping Coefficient

During vibration of the foundation, energy is transmitted away to the soil by the spreading upwards-downwards waves and shear waves [G1]. The upwards-downwards propagating waves are close to the Lysmer's analog velocity rather than P-waves and the sidewalls mainly transmit shear waves (S-wave) to the surrounding soil. The damping coefficient is frequency dependent and increases with the contact area of the soil-foundation. By assuming that the two waves generated at the base and sides of the embedded foundation are independent, the total radiation damping C_v can be calculated by

$$C_v = \rho\, V_{La}\, A_b\, c_z + \rho\, V_s\, A_s \tag{3.33}$$

in which

$$V_{La} = \frac{3.4 \cdot V_s}{\pi\,(1 - \nu)} \tag{3.34}$$

where ρ is the soil mass density, V_{La} is Lysmer's analog wave velocity, V_s is shear wave velocity, A_b and A_s are area of the foundation base and the soil-sidewalls contact area respectively, c_z is a frequency and shape dependent coefficient.

3.5.2 Horizontal Stiffness and Damping Coefficients

a. Horizontal Static Stiffness Coefficient

The dynamic response of arbitrary shaped fully or partially embedded foundations have been presented by Gazetas and Tassoulas (1987). The soil-foundation system is shown in Fig. 3.9. From the study of the dynamic response of an arbitrary shaped surface foundation [D1], the horizontal static stiffnesses of the surface foundation in the x-direction $K_{sx,sur}$ and in the y-direction $K_{sy,sur}$ can be respectively calculated by

$$K_{sy,sur} = \frac{2\,G\,L}{(2 - \nu)}\, S_y \tag{3.35}$$

$$K_{sx,sur} = K_{sy,sur} - \frac{0.21}{(0.75 - \nu)} G L \{1 - (\frac{B}{L})\} \tag{3.36}$$

and the value of S_y is dependent on the base-shape parameter and can be expressed as

$$S_y = 2 + 2.5\{\frac{A_b}{4 L^2}\}^{0.85} \tag{3.37}$$

The schematic effects of the trench and sidewall of the embedment on the foundation response are similar to those discussed in Section 3.5.1. Results for rectangular foundations indicate that both trench coefficient, I_{tre} and sidewall coefficient, I_{wall} are independent of the loading directions. These lead to

$$I_{y,tre} = I_{x,tre} = I_{tre} \tag{3.38}$$

$$I_{y,wall} = I_{x,wall} = I_{wall} \tag{3.39}$$

however, I_{tre} is affected by a single parameter D/B and I_{wall} is affected by dimensionless parameters h/B and A_w/L^2 [G2]. Both I_{tre} and I_{wall} can be written as

$$I_{tre} = 1 + 0.15\sqrt{\frac{D}{B}} \leq 1.20 \tag{3.40}$$

$$I_{wall} = 1 + 0.52 \{\frac{h}{B} \frac{A_w}{L^2}\}^{0.4} \tag{3.41}$$

where A_w is the effective contact area of the soil sidewall and h is the distance from the mid-height of the sidewall to the ground surface.

The static horizontal stiffness of an arbitrary shaped embedded foundation can be computed by considering both coefficients in Eqs. 3.40 and 3.41 then

$$K_{sy,emb} = K_{sy,sur} I_{tre} I_{wall} \tag{3.42}$$

$$K_{sx,emb} = K_{sx,sur} I_{tre} I_{wall} \tag{3.43}$$

b. Horizontal Dynamic Stiffness Coefficient

The effects of the excitation frequency on the dynamic horizontal stiffness of the embedded foundation are represented by the dynamic horizontal stiffness coefficients $k_{x,emb}$ and $k_{y,emb}$ which can be determined by considering the effect of the embedment aspect ratio [G4]. The horizontal dynamic stiffnesses are

$$K_{dy,emb} = K_{sy,emb}\ k_{y,emb} \tag{3.44}$$

$$K_{dx,emb} = K_{sx,emb}\ k_{x,emb} \tag{3.45}$$

where $K_{dy,emb}$ and $K_{dx,emb}$ are the dynamic horizontal stiffnesses in the y and x-directions.

c. Horizontal Damping Coefficient

The horizontal damping coefficient of an embedded foundation is initially generated by both shear waves V_s and compression-extension waves V_{ce}. The shear waves initiated at the foundation base propagate away to the surrounding soil with velocity V_s. The equivalent energy dissipated at the foundation base can be expressed by

$$C_{bx} = c_x\ \rho\ A_b\ V_s \tag{3.46}$$

$$C_{by} = c_y\ \rho\ A_b\ V_s \tag{3.47}$$

where C_{bx} and C_{by} are the damping effects generated by a shear wave at the foundation base in the x and y-directions respectively and c_x and c_y are frequency and shape dependent coefficients in the x and y-directions respectively. Both these radiation damping coefficients are functions of the dimensionless frequency, the L/B ratio and Poisson's ratio of the soil [G4]. It was found that an approximate value of 1 for c_x can be used for practical applications.

The energy dissipation generated by the sidewalls C_w is due to shear waves V_s propagating in a direction perpendicular to the horizontal motions and compression-extension waves with velocity V_{ce} propagating in direction of the horizontal motions. The total energy dissipated by sidewalls C_w can be computed by

$$C_w = c_s\ (\rho\ A_{ws}\ V_s\ ) + c_{ce}\ (\rho\ A_{wce}\ V_{ce}\ ) \tag{3.48}$$

where c_s and c_{ce} are the dimensionless damping coefficients referring to the propagating directions of V_s and V_{ce} respectively, A_{ws} is the soil-sidewall contact area parallel to the horizontal motions and A_{wce} is the soil-sidewall contact area perpendicular to the horizontal motions. Numerical results suggests that for D/B less than 2, an approximate value of 1 for c_s and c_{ce} can be used for practical applications.

Assuming that the foundation base and sidewalls radiate energy independently, total energy dissipation of the embedded foundation in term of radiation damping coefficients is

$$C = C_b + C_w = (\rho\ A_b V_s\ ) + (\rho\ A_{ws} V_s + \rho\ A_{wce} V_{ce}) \tag{3.49}$$

3.5.3 Effects of Material Damping

As mentioned in Section 3.4.1, the damping coefficients on the soil-foundation model include not only radiation damping but also material damping. It is also assumed that the material damping is independent of the soil Poisson's ratio. The effect of soil material damping ξ_m on the shear wave velocity $V_{s,m}$ can be expressed in complex form as

$$V_{s,m} = \frac{V_s}{(1 - i\ \xi_m)} \tag{3.50}$$

and the complex soil shear modulus G_{cm} can be written as

$$G_{cm} = G\ (1 + 2\ i\ \xi_m) \tag{3.51}$$

where $V_{s,m}$ is shear wave velocity taking into account of the effect of soil material damping, V_s is the shear wave velocity and G is the shear modulus of the soil. The dynamic impedance of the soil-foundation interaction can be written as

$$K_d = K_s\{k(a_0) + i\ a_0\ c\ (a_0)\}\ (1 + 2\ i\ \xi_m) \tag{3.52}$$

where K_d and K_s are the dynamic and static stiffnesses respectively, k and c are the dimensionless dynamic stiffness and damping coefficients of the vibration modes and a_0 is dimensionless frequency.

The effects of soil material damping ξ_m on the soil-foundation stiffness and damping coefficients are discussed in Refs. D2, G1, G2 and G3. These effects on the dynamic stiffness $K_d(\xi_m)$ and damping coefficient $C_d(\xi_m)$ can be expressed by

$$K_d\ (\xi_m) = K_d - \omega\ C_d\ \xi_m \tag{3.53}$$

$$C_d\ (\xi_m) = C_d + \frac{2\ K_d}{\omega}\ \xi_m \tag{3.54}$$

where C_d is the dynamic damping coefficient and ω is the natural frequency of the structure.

3.5.4 Soil-Foundation Impedance

The lumped parameter method is used to analyse soil-structure interaction based on the frequency dependent soil-foundation impedance with a constant impedance coefficient. In this method, the soil-foundation impedance is represented by a single spring and dashpot to simulate the soil-foundation stiffness and radiation damping respectively. In general, the soil may not be purely cohesive or purely cohesionless but possesses both an angle of internal friction ϕ and cohesion c. This type of soil is usually called a c-ϕ soil and is assumed in this study.

The parameters used in calculations of the soil-foundation impedances are presented for the following: the soil cohesion c = 0.7 kg/cm^2, the angle of internal friction ϕ = 22°, the soil mass density ρ = 1800 kg/m^3, the frequency of excitation ω = 15.71 rad/sec, the Poisson's ratio of the soil ν = 0.38, the shear wave velocity V_s = 150 m/sec. In this study, the soil-foundation impedances are based on the soil-shear modulus $G = 0.70\ G_{max}$ (420kg/cm^2) corresponding to a soil shear strain $\gamma_{ss} = 4 \times 10^{-4}$. A simple computer program was written to calculate the stiffness and damping coefficients of partially embedded foundations [W1] as discussed in Section 3.5. The soil-foundation impedance was based on the soil-mechanical model as expressed in Section 3.4.1. Effects of the material damping on the foundation impedance were based on Eqs. 3.53 and 3.54 as discussed in Section 3.5.3.

3.6 Building Frame Models Used in This Study

As mentioned previously, a 12-storey reinforced concrete building prototype was used in this study. The two bay symmetric frame with a beam span of 9.2 m was designed as a plane frame according to two different design standards. One building was originally designed by Jury (1978) [J4] and subsequently modified based on NZS 3101:1982 [N1] and the second one was designed to NZS 3101:1995 [S9]. The main difference for the buildings designed to two design standards is based on different choice of structural ductility. The structural ductilities are respectively 4.0 and 5.0 for the structures designed to NZS 3101:1982 and NZS 3101:1995. The two different buildings are designed in accordance with the provisions of the loading code NZS 4203:1992 [S8]. The interstorey height is 5.0 m for the ground floor and 3.65 m for the upper floors. As the structures were moment-resisting frames, shear deformation of the members was not taken into account. As the ductility of the base isolated building is approximately 1 and the responses were almost elastic during the time history analyses. The complete properties of the two different structures are listed in Appendix D.

One of the problems in the use of conventional base isolation in buildings is that under some excitations, the displacements required in the isolation system are large and may not be readily available in many current devices. One suggested solution by Cui (1995) [C13] is to distribute the devices through the height of the building, the devices at each level providing part of the required displacement and limiting the dislocation of services, etc., at each isolation level. This study also seeks to evaluate the effect of using this segmental building isolation system where the isolation devices are placed at various levels in the building in order to reduce the displacements imposed on each of the devices.

Prototypes of the structural systems were used in this study. The fixed base model is assumed to be fixed at the footing level as shown in Fig. 3.10. In the base isolated buildings, the isolation devices are placed at the base of structure and the frame is assumed to have either a rigid base or a compliant foundation using a Voigt-Kelvin model as shown in Figs. 3.11 and 3.12 respectively. In the case of the segmental structure, the building is divided into three segments that are interconnected by conventional isolation systems at two upper storeys as well as at the base of the frame. The segmental models are assumed to have either a rigid base or to allow foundation compliance using a Voigt-Kelvin model as shown in Figs. 3.13 and 3.14 respectively.

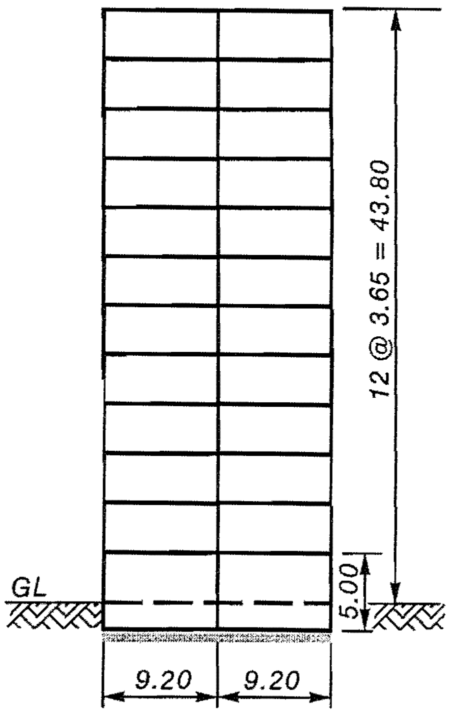


Fig. 3.10 Fixed Base Model

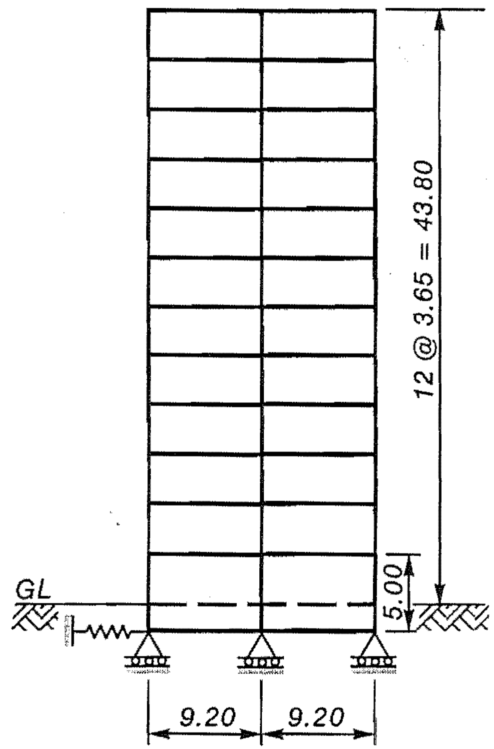


Fig. 3.11 Base Isolated Model on a Rigid Base

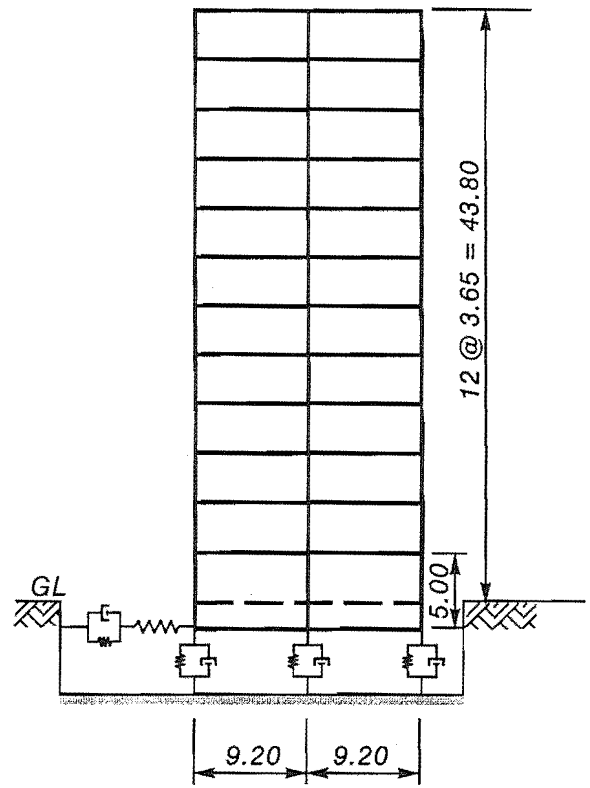


Fig. 3.12 Base Isolated Model on a Compliant Foundation

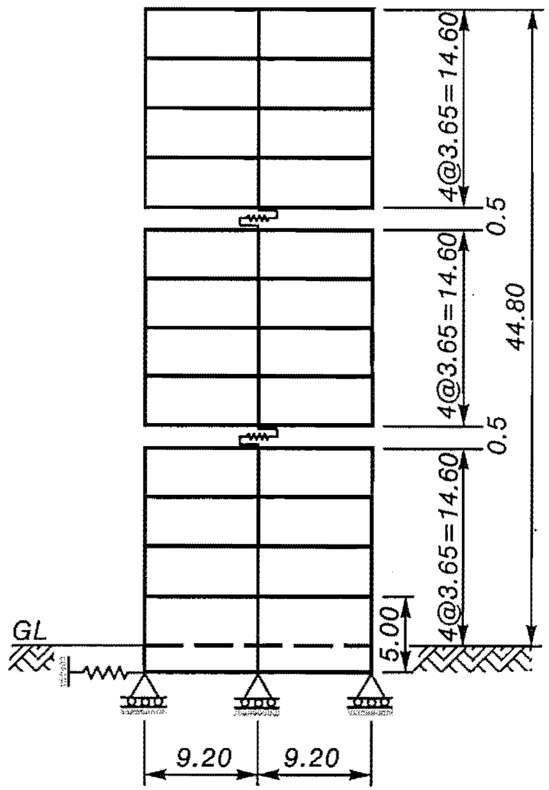


Fig. 3.13 Segmental Model on a Rigid Base

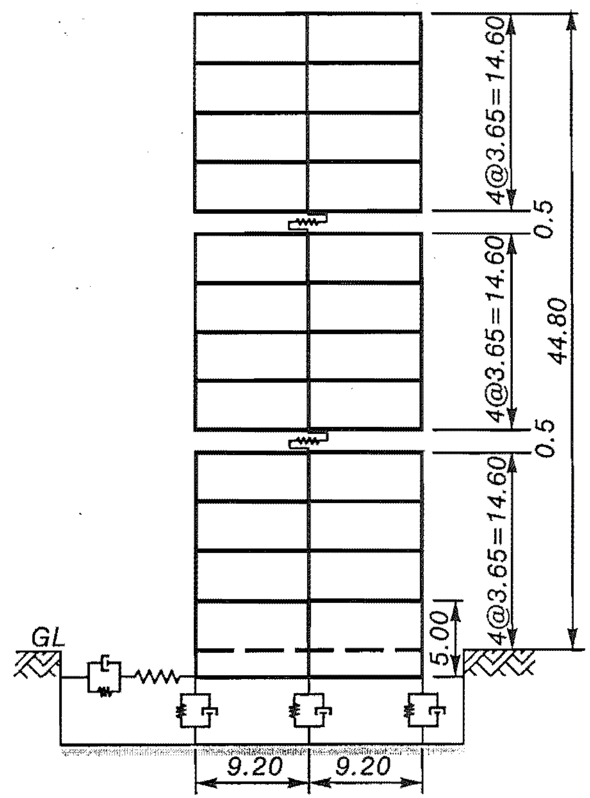


Fig. 3.14 Segmental Model on a Compliant Foundation

The main purpose of this study will be to investigate the seismic responses of the base isolated and segmental buildings with a rigid base and a compliant foundation. As introduced in Fig. 3.10, the fixed base model is used as a reference structure. For the segmental models shown in Figs. 3.13 and 3.14, rigid links will be implied between the two segments to prevent the occurrence of rocking of the structure and to transmit the gravity loads between the two segments. The details of the superstructure model with uniform mass and stiffness over the height of the superstructure will be shown in Appendix E.

CHAPTER 4

ANALYSIS PROCEDURES

4.1 Introduction

This study will evaluate and compare the responses of base isolated buildings with those of fixed base structures. It also seeks to evaluate the effect of using segmental structures [C13] where isolation devices are placed at various heights in the structure, as well as at the base, in order to reduce the displacements imposed on each of the devices. The above responses do not include base rocking motions during the seismic analyses of structures, apart from that associated with the foundation compliance.

The understanding of the characteristics of the isolation systems and simulation of their nonlinear hysteretic behaviour presented in Section 3.4.2 is an important part of the analysis of base isolated buildings. The superstructure modelling was illustrated in Section 3.4.3. Another aspect is the refinement of the soil-foundation modelling and impedance as discussed in Sections 3.4.1 and 3.5 respectively. Studies into other aspects affecting the response of base isolated structures will be briefly discussed in this chapter.

4.2 The Soil Site Modelled in This Study

In seismic regions, geotechnical site investigations obviously should include the gathering of information about the physical nature of the site and its environs that will allow an adequate evaluation of seismic hazard to be made. In order to obtain the maximum benefit from any method of seismic analysis, an understanding of the dynamic response characteristics of the material is essential because some soils increase in strength under rapid cyclic loading, while others, such as saturated sands or sensitive clays, may lose strength with vibration [D3]. Often, the seismic analysis of structures assumes that the structure is located on a rock or very stiff soil site which gives a considerable underestimation of the responses of buildings because it is actually sited on a softer foundation. Thus, the subsoil site plays an important role on the seismic analysis of the structures if an appropriate estimation of responses is to be obtained.

In order to evaluate the seismic response of a structure at a given site, the dynamic properties of the combined soil-structure system must be understood. The nature of the subsoil may influence the response of the structure in the following ways:

1. The seismic excitation at bedrock is modified during transmission through the overlying soils to the foundation.

2. The fixed base dynamic properties of the structure may be significantly modified by the presence of soils overlying bedrock. This will include changes in the mode shapes and periods of vibration.

3. A significant part of the vibrational energy of the flexibly supported structure may be dissipated by material damping and radiation damping in the supporting medium.

4. Structures sited on soft alluvium may be damaged by differential vertical displacements occurring before and/or during earthquakes. This effect is in contrast to resonance which, in the case of soft ground, will of course occur for longer period structures [D3,S1].

For this study the chosen design spectrum is the intermediate soil site spectrum from NZS 4203:1992 [S8]. This spectrum does not diminish as rapidly as the period increases as does the spectrum for hard rock sites.

4.3 Comparison of Earthquake and Wind Loading

As introduced in Section 3.6, the structural models designed to NZS 3101:1982 [N1] and NZS 3101:1995 [S9] are used to calculate the base shear forces in the structure induced by an earthquake using the equivalent static method, and these are compared with those forces induced by wind loading according to NZS 4203:1992. The ratios of earthquake design base shear force to the weight of the 12-storey building structure for different structural ductilities were compared with the associated wind load are shown in Table 4.1. From the results, it is seen that the earthquake-induced base shear force is greater than the wind-induced base shear force so that

Load Type	Ratio of Base Shear to Weight of the Structure						
	Serviceability Limit State	Ultimate Limit State					
	μ=1	μ=1	μ=2	μ=3	μ=4	μ=5	μ=6
Wind Load	2.0%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%
Earthquake Load	3.3%	20.0%	10.4%	6.6%	5.1%	4.0%	3.4%

(a) Structures Designed to NZS 3101:1982

Load Type	Ratio of Base Shear to Weight of the Structure						
	Serviceability Limit State	Ultimate Limit State					
	μ=1	μ=1	μ=2	μ=3	μ=4	μ=5	μ=6
Wind Load	2.4%	3.2%	3.2%	3.2%	3.2%	3.2%	3.2%
Earthquake Load	2.7%	16.0%	8.0%	5.2%	4.0%	3.2%	2.7%

(b) Structures Designed to NZS 3101:1995

Note:

μ is the lateral structural ductility of the building.

Table 4.1 Comparisons of Base Shear to Weight of 12-Storey Building by Wind-induced and Earthquake-induced Loads base isolation devices can be used as a feature of the seismic design of the structures. Also, the isolator yield force is required to exceed the level of the wind loading used in design by a margin to prevent yield under wind-storm conditions.

4.4 Selection of Base Isolation System

The successful seismic isolation in the design of a seismically isolated structure depends strongly on the selection of an appropriate base isolation system. This will partly be governed by the nature of the design criteria. It is important to have a deep understanding of the influence of each parameter controlling the behaviour of the isolation device on the performance of the base isolated building during earthquakes. The selection of base isolation devices is generally decided by two steps; at the initial design stage, it is necessary to consider whether the addition of seismic isolation will prove to be a cost-effective means of providing appropriate levels of seismic resistance for a structure and its significant secondary structures and contents, the final decision to use seismic isolation must be made on a case-by-case basis.

Seismic isolation below the structure provides flexibility which generally reduces the severity of earthquake attacks. For the design of the base isolated building, Skinner et al (1993) [S6] mentioned that the isolator deformations and structural displacements must be considered and accepted in order to achieve the reductions in seismic response. Seismic isolation gives several benefits; firstly, isolation gives a large increase in the first-mode period and this may sometimes be used to reduce severe seismic response of the structure if the severity is caused by approximate tuning to the period of an unisolated structural first mode; secondly, hysteretic isolators may be used to confer ductility to brittle structures, thus enabling them to resist seismic loads and if the structure has high stiffness and low damping, effective ductility can be introduced without large increases in structural deformations.

For the purpose of controlling the base displacement and resisting the wind load or small base disturbance, most of the base isolation systems are designed to have non-linear hysteretic characteristics. The non-linearity also allows the structure to be stiff enough to resist wind load and minor base excitation, while in strong ground motions, to be soft enough to provide the large base flexibility required for effective isolation. As discussed in Skinner et al (1993), the nonlinear isolation systems can usually produce lower values of first-mode dominated response quantities such as base shears and displacements, while linear systems are particularly effective at suppressing high-frequency responses. As mentioned in Section 3.4.2, nonlinear hysteretic isolation systems is assumed in this study for the base isolated structures.

4.5 Dynamic Parameters of Base Isolated Structures

The three key parameters of a base isolation system, i.e. the initial stiffness, the post-yield stiffness and the yield strength, have an important role in determining the effective stiffness and energy dissipation capacity of the base isolation system, which in turn governs the structural response. The dynamic characteristics and seismic response of the structure to the 1940 El Centro N-S earthquake are investigated.

First, an important parameter required to define a nonlinear isolation system as discussed in Section 3.4.2 is the yield ratio F_y/W, relating the yield force F_y of the isolator to the weight W of the structure. Skinner et al (1993) proposed that for a design earthquake having the severity and character of the El Centro N-S 1940 accelerogram, a yield ratio F_y/W of approximately 5% usually gives suitable values for the isolator forces and displacements. In this study, the value of F_y/W is determined by comparisons of base shear of the structure with earthquake-induced and wind-induced forces as shown in Table 4.1. Therefore, the yield strengths, F_y, are respectively 3% and 5%W for structures designed to NZS 3101:1995 and NZS 3101:1982, where W is the total weight of the structure.

Secondly, k_o is varied from 2.5 W/m to 25.0 W/m, while αk_o is kept constant at 0 and 1.25 W/m, and F_y/W is used as mentioned above. The effects of these parameter variations on the base shear and base displacement using the time history analyses are listed in Table 4.2. It can be seen that the base shears reach their minimum values when the initial stiffnesses, k_o are 5.0 and 10.0 W/m at post-yield stiffness, αk_o of 0 and 1.25 W/m respectively. Also, a base isolation system with a stiffer k_o tends to minimize the base displacement. This is true for the structures designed to NZS 3101:1982 and NZS 3101:1995. A base isolator reaches its optimum performance in reducing not only the base shear but also the base displacement. Therefore, the initial stiffness k_o of the base isolator of 10 W/m is suggested as an optimum value in this study. The post-yield stiffnesses, αk_o, are taken as 0 and 0.1 to 1.5 W/m, i.e. $\alpha = 0$ and 0.01 to 0.15 respectively for the elasto-plastic and bilinear models of the isolation systems.

αk_o (W/m)	F_y (%W)	k_o (W/m)	Base Shear / Weight of the Structure	Base Floor Displacement (m)
1.25	5	2.5	0.0982	0.076
		5.0	0.0968	0.062
		10.0	0.0884	0.052
		25.0	0.0901	0.050
0	5	2.5	0.0612	0.096
		5.0	0.0592	0.090
		10.0	0.0601	0.068
		25.0	0.0673	0.063

(a) Structures Designed to NZS 3101:1982

αk_o (W/m)	F_y (%W)	k_o (W/m)	Base Shear / Weight of the Structure	Base Floor Displacement (m)
1.25	3	2.5	0.0792	0.072
		5.0	0.0764	0.060
		10.0	0.0691	0.047
		25.0	0.0695	0.042
0	3	2.5	0.0482	0.092
		5.0	0.0442	0.083
		10.0	0.0453	0.066
		25.0	0.0488	0.058

(b) Structures Designed to NZS 3101:1995

Table 4.2 The Effects of Varying Initial Stiffness on Base Shears and Base Floor Displacements

4.6 Choice of the Earthquake Input

The earthquake ground motion is an important parameter in earthquake resistant design of base isolated structures. Usually, the earthquake resistant design of buildings depends on two aspects: firstly, the characteristics of the earthquake such as the frequency of occurrence, intensity, magnitude and ground acceleration and secondly, the design criteria depending on whether the building is designed for the serviceability or ultimate limit state [B2,P5]. In principle, design earthquakes for seismically isolated structures should be selected on the same general basis as design earthquakes for an unisolated structure at the same site. Appropriate return periods for design level and extreme or maximum credible motions are selected on a similar basis to those for unisolated structures, taking into account the seismicity of the region and the importance and risk factors for the structure.

The peak ground acceleration plays an important role in the selection of the design earthquake even though it provides a poor estimation of the damage potential, especially for a relatively flexible structure [P5,T2]. Damage observed, and calculated, due to earthquakes indicate that the ground velocity [M3,P5] and acceleration to velocity ratio [T2,Z1,Z2] provide better estimation than ground acceleration. Some attempts have also been made to correlate the structural response with some ground motion characteristics. Zhu, Tao and Heidebrecht [Z1,Z2] studied the effect of peak ground acceleration to velocity ratio on the ductility demand of inelastic systems in order to incorporate this parameter in the specification of seismic design base shear of designed unisolated structures. It should be realized that many lessons must still be learned from the occurrence of recent and future ground motions before the importance of ground shaking parameters used in predicting the structural behaviour can be fully understood [E1].

In general, many seismic codes worldwide have used the earthquake record, El Centro 1940, as a basis for seismic resistant design criteria. This earthquake was centred along a fault of the San Andreas fault system in southern California and had an average local Richter magnitude, M=6.4 and a complex pattern of energy release with a series of multiple ruptures moving generally south-eastwards over a distance of 25 km away from the epicentre. The record from the El Centro site has a special significance for earthquake engineering since it was the first strong motion ever recorded in the epicentral region of a moderate sized earthquake.

For many sites with high seismicity, and ground of moderate flexibility and high strength, El Centro-like accelerograms and spectra may be used for seismic design. In this study, a 20-second duration of the N-S component of the El Centro 1940 record is used for the analyses of base isolated structures on rigid or intermediate soil sites. At the same time, earthquakes with a low and high excitation frequencies are also considered. There are four real earthquake records that will be used for the dynamic analyses in this study. Each earthquake record has different characteristics. El Centro 1940 N-S has many peaks with a few pulses at the commencement of the earthquake, Taft 1952 N69W has many peaks of a similar magnitude, Parkfield 1966 N65E has a large pulse over a very short time duration, and Mexico 1985 SCT S00E has the strongest excitation at the long periods. The main purpose is to investigate the behaviour of base isolated and segmental multistorey structures using the time history analyses under different types of ground motions.

In this study, the 12-storey multistorey structures will be used during the time history analyses. As introduced above, the Parkfield 1966 N65 E ground motion show a large pulse over a very short time duration and has some of the "fling" characteristics of a near-fault earthquake. This is similar to the effects observed in the 1994 Northridge and the 1995 Kobe earthquakes.

4.7 The Method of Analysis Used in This Study

In this study, the methods of seismic analysis for the base isolated and segmental structures have been discussed. The lumped parameter method is used to analyse the soil-foundation modelling and impedance as discussed in Sections 3.4.1 and 3.5 respectively. The bilinear and elasto-plastic models are used to simulate the nonlinear hysteretic behaviour of various base isolation systems as discussed in Section 3.4.2. Also, the frame structure deforming in a shear-like manner behaves linearly elastically during the design level earthquake as illustrated in Section 3.4.3. The building models are discussed in Section 3.6, the details of the superstructure models are shown in Appendix E. These building models are analysed using the computer program *Ruaumoko* which is briefly presented below.

The computer program *Ruaumoko* for the seismic analyses was written initially by Sharpe [S3] and extensively modified and developed by Carr [C1] over the past two decades. It was designed to produce a step-by-step time-history response of a non-linear two-dimensional general frame structure subjected to an horizontal and/or a vertical earthquake accelerogram. If required, the program carries out a static analysis for the structure, then the program performs a free vibration modal analysis to calculate the natural periods and mode shapes of free vibration of the structure before conducting the inelastic dynamic time history analysis. Many options are available for modelling members, hysteresis rules, mass matrices and damping matrices. It also gives the user options for considering or neglecting P-delta effects.

CHAPTER 5

THE SEISMIC RESPONSES OF BASE ISOLATED STRUCTURES SUBJECTED TO THE 1940 EL CENTRO N-S EARTHQUAKE

5.1 Introduction

The benefit of implementing a base isolation device is to protect the structure from seismic attack. In this chapter, the seismic responses of 12-storey frame structures deforming in a shear-like manner mounted on nonlinear isolation systems subjected to the N-S component of El Centro 1940 earthquake will be investigated.

The base isolated buildings are assumed to either have a rigid base or allow foundation compliance as discussed in Sections 3.4 and 3.5, and the computer program *Ruaumoko* [C1] is utilised as a tool for conducting the inelastic time history analysis. In the time history analyses, the uniform superstructure model is used as introduced in Section 3.4.3, and the elasto-plastic and bilinear hysteretic models are used to simulate the force-displacement relationship of the nonlinear base isolation systems.

As discussed in Section 3.6, the 12-storey reinforced concrete buildings were designed as plane frames according to two different design standards. One building was designed to NZS 3101:1995 [S9] and the second one was originally designed by Jury [J4] and extensively modified based on NZS 3101:1982 [N1], and in accordance with loading code NZS 4203:1992 [S8]. Further, the seismic responses for other types of 12-storey buildings, including an unisolated one on a fixed base and segmental ones with rigid bases and compliant foundations as proposed by Cui (1995) [C13] where the isolators placed on the various levels as well as at base, are compared in order to show the typical performance of these base isolated structures.

5.2 Dynamic Parameters of Nonlinear Models

As mentioned by Andriono (1990) [A2], the more significant the higher mode contributions compared to the contribution of the first mode, the more bulged is the lateral shear envelope. He also indicated that the structure response at the base is strongly governed by the first mode whose period is lengthened by the base isolation system when the system yields, but the response in the middle to top floors may have a significant influence from the higher modes. In general, the question is how to measure these modal contributions. Andriono (1990) used quantitatively approximate measurements to gain some insight into the structural behaviour and these are beneficial in designing the base isolated structures.

In this study, the isolator yield strengths for the elasto-plastic and bilinear models are required to exceed the level of the wind loading used in design by a margin to prevent yield under wind-storm condition. When carrying out a time history analysis for the segmental structure, the initial stiffness of the isolator affects the lateral displacements between the two adjacent segments of the structure. The deflection limit is based on the isolator displacement capacity. The deflection limit also depends on the limits posed by the actual structural layout so as to reduce the structural and non-structural damage in the levels containing the isolators.

From Table 4.1, the yield strengths of the base isolation systems to the weights of the structures, F_y/W, are respectively taken as 3% and 5% for the buildings designed to NZS 3101:1995 and NZS 3101:1982. As discussed in Section 4.5, the initial stiffness, k_o, of the base isolation system is ten times the total weight of the structure per metre (10.0 W/m).

In order to determine an appropriate post-yield stiffness, αk_o, for the bilinear isolation system, the base shears of the base isolated and segmental structures are used to justify whether the isolation system is beneficial or not when compared with those of the fixed base buildings as shown in Table 5.1. From the results shown in Table 5.1 (a), the greater the post-yield ratio, α, the greater is the base shear of the base isolated structure designed to NZS 3101:1995 and NZS 3101:1982. Similar results are also seen in the segmental building as shown Table 5.1 (b).

From Table 5.1 (a), for a post-yield ratio, α, of 0.05 or less, the base shear of the base isolated building is smaller than that of the fixed base building. Figs. 5.1 (b) and 5.2 (b) show a regression of post-yield ratio, α, from 0.01 to 0.05 for the bilinear models of the base isolated buildings designed to NZS 3101:1995 and NZS 3101:1982 using an approximate measurement. As can be seen from the Figs. 5.1 (b) and 5.2 (b), the post-yield ratio of 0.04 is suggested as an optimum for the base isolated building.

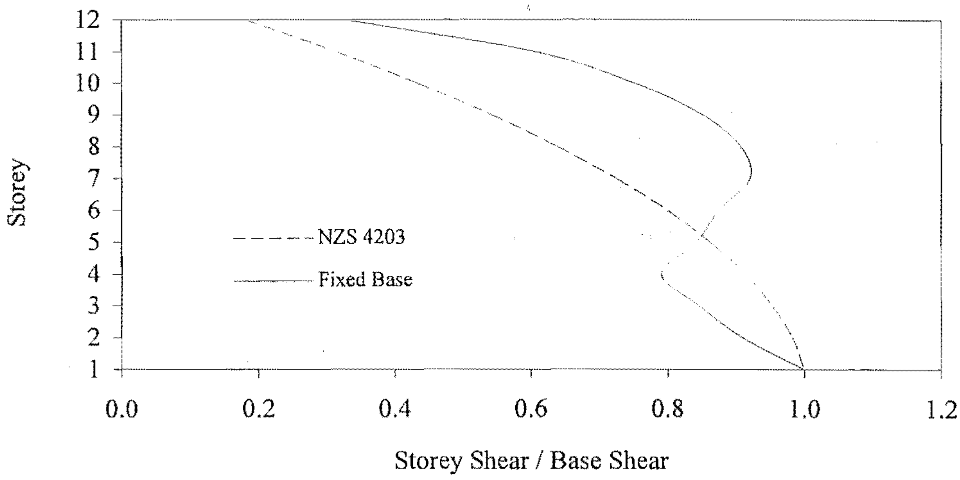
Type	Structures Designed to NZS 3101:1995			Structures Designed to NZS 3101:1982		
	Base Shear / Weight of Structure			Base Shear / Weight of Structure		
Fixed Base	0.0675			0.0812		
Base Isolated	F_y (%W)	α	Base Shear /Weight of Structure	F_y (%W)	α	Base Shear /Weight of Structure
	3.0	0.150	0.0731	5.0	0.150	0.0960
		0.125	0.0710		0.125	0.0896
		0.100	0.0688		0.100	0.0832
		0.075	0.0679		0.075	0.0809
		0.050	0.0603		0.050	0.0732
		0.040	0.0545		0.040	0.0681
		0.030	0.0506		0.030	0.0647
		0.020	0.0472		0.020	0.0615
		0.010	0.0462		0.010	0.0609
		0.000	0.0453		0.000	0.0601

(a) Comparisons of Fixed Base and Base Isolated Buildings

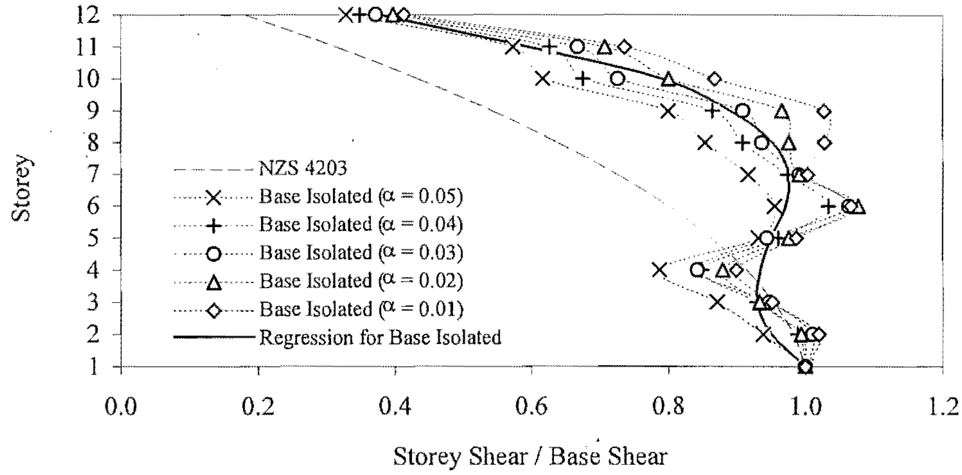
Type	Structures Designed to NZS 3101:1995			Structures Designed to NZS 3101:1982		
	Base Shear / Weight of Structure			Base Shear / Weight of Structure		
Fixed Base	0.0675			0.0812		
Segmental	F_y (%W)	α	Base Shear /Weight of Structure	F_y (%W)	α	Base Shear /Weight of Structure
	3.0	0.150	0.0557	5.0	0.150	0.0711
		0.100	0.0525		0.100	0.0694
		0.050	0.0404		0.050	0.0591
		0.000	0.0373		0.000	0.0575

(b) Comparisons of Fixed Base and Segmental Buildings

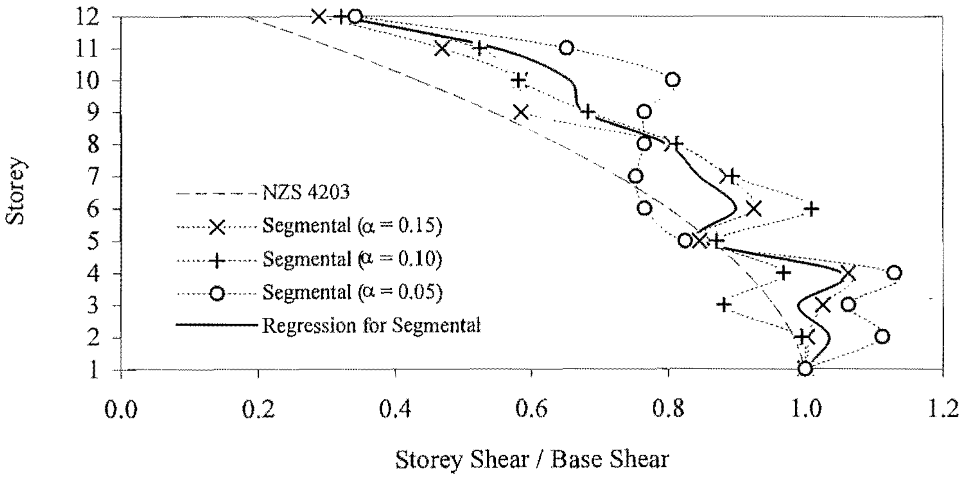
Table 5.1 Base Shears for Different Structures of the Time History Analyses



(a) Fixed Base Building

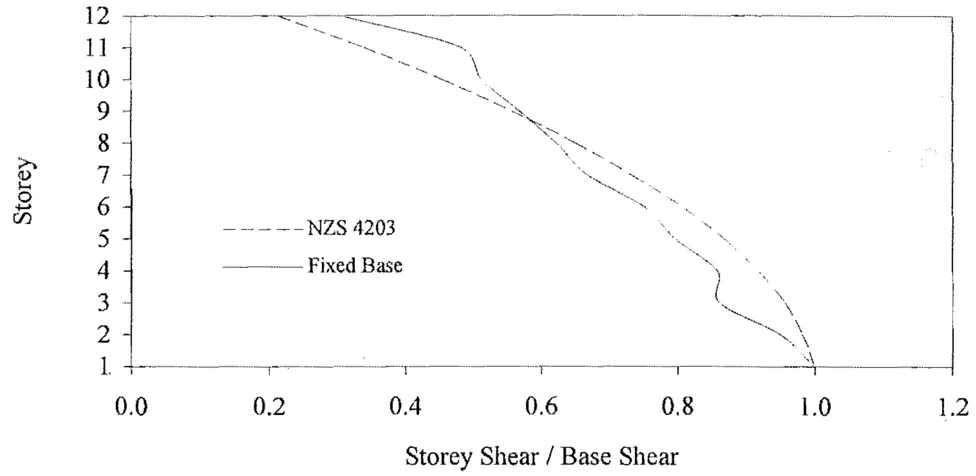


(b) Base Isolated Building

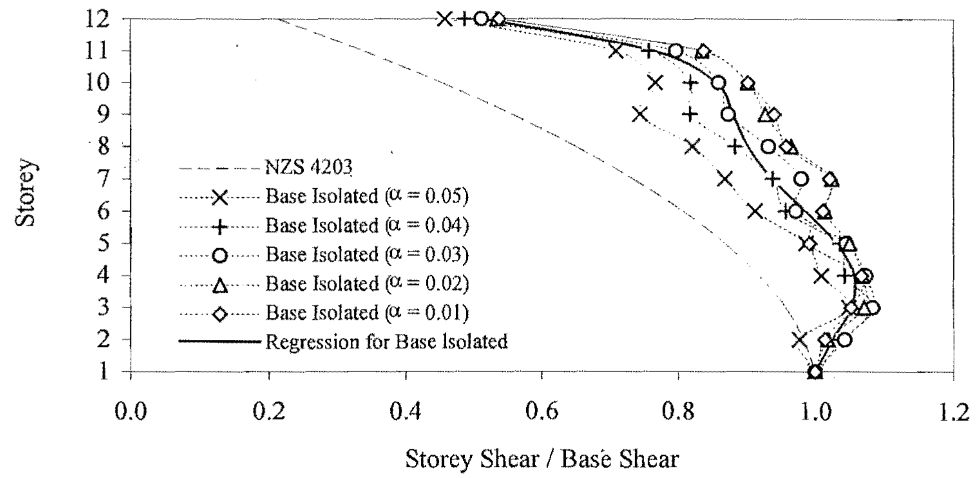


(c) Segmental Building

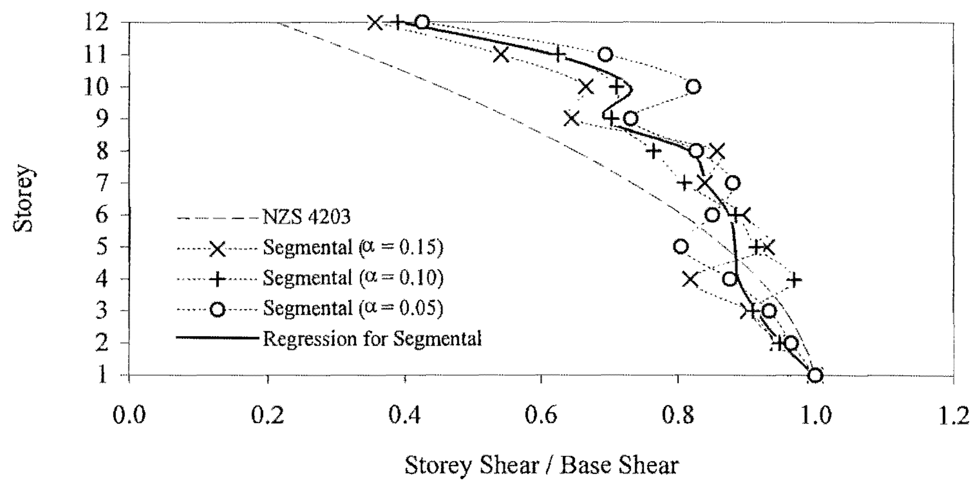
Fig 5.1 Normalized Lateral Shear Envelopes for Structures Designed to NZS 3101:1995



(a) Fixed Base Building



(b) Base Isolated Building



(c) Segmental Building

Fig 5.2 Normalized Lateral Shear Envelopes for Structures Designed to NZS 3101:1982

As shown in Table 5.1 (b), for a post-yield ratio, α, of 0.05 to 0.15, the base shears of the segmental structures are smaller than those of the fixed base buildings. Using a similar method to that mentioned above, the regression of post-yield ratio, α, from 0.05 to 0.15 for the bilinear models of the segmental buildings designed to NZS 3101:1995 and NZS 3101:1982 are shown in Figs. 5.1 (c) and 5.2 (c) respectively. From the results shown in Figs. 5.1(c) and 5.2 (c), and the maximum beam curvature ductility demands shown in Table 5.2 (a) and (b), the post-yield ratio of 0.05 is suggested as an optimum for the segmental building.

Finally, the three key parameters of a base isolation system are used below. The initial stiffness, k_o, is taken as ten times the total weight of the structure per metre (10.0 W/m). The post-yield stiffnesses, αk_o, are taken as 0 and 0.4 W/m, i.e. $\alpha = 0$ and 0.04 for the elasto-plastic and bilinear models in the base isolated structures. For the segmental structures, αk_o are taken as 0 and 0.5 W/m, i.e. $\alpha = 0$ and 0.05 for the elasto-plastic and bilinear models. The yield strengths, F_y, are respectively 3% and 5%W for the structures designed to NZS 3101:1995 and NZS 3101:1982, where W is the total weight of the structure.

5.3 Fundamental Periods of Structures

In a structural dynamic analysis, the fundamental period is used to describe the stiffness of the structure and it is also useful for preliminary analysis. In the equivalent static method specified by NZS 4203:1992, the base shear is calculated from the structural fundamental period; therefore the magnitude of the design force depends upon this period. The most important feature of seismic isolation is that increased flexibility increases the fundamental period of the structure. Because the period is increased beyond that of the peak acceleration response of the earthquake, resonance is avoided and the seismic acceleration response is also reduced [S6].

The fundamental periods of the different 12-storey buildings are shown in Table 5.3. From this table, it can be seen that for the structures designed to NZS 3101:1995, the fundamental periods of the base isolated and segmental buildings with a compliant foundation are 5.7% and 5.8% longer than those with a rigid base. For the structures designed to NZS 3101:1982, it was found that the fundamental periods of the base isolated and segmental buildings with a compliant foundation are 11.7% and 11.3% longer than those with a rigid base. These results imply that due to the foundation flexibility, the base isolated and segmental structures with a compliant

Storey	Maximum Beam Curvature Ductility Demands			
	$\alpha = 0.15$	$\alpha = 0.10$	$\alpha = 0.05$	$\alpha = 0.00$
12	1.08	< 1.00	< 1.00	< 1.00
11	2.45	2.45	2.18	1.96
10	2.54	2.58	2.59	2.29
9	2.52	2.54	2.56	2.31
8	1.17	< 1.00	< 1.00	< 1.00
7	2.54	1.54	< 1.00	< 1.00
6	1.75	1.01	< 1.00	< 1.00
5	1.48	< 1.00	< 1.00	< 1.00
4	1.42	< 1.00	< 1.00	< 1.00
3	2.82	1.94	1.13	< 1.00
2	2.89	1.99	1.01	< 1.00
1	2.26	1.31	< 1.00	< 1.00

(a) Structures Designed to NZS 3101:1995

Story	Maximum Beam Curvature Ductility Demands			
	$\alpha = 0.15$	$\alpha = 0.10$	$\alpha = 0.05$	$\alpha = 0.00$
12	2.30	2.35	2.48	2.85
11	3.38	3.31	3.29	2.98
10	1.67	1.71	1.55	1.42
9	1.65	1.44	1.24	1.62
8	< 1.00	< 1.00	< 1.00	< 1.00
7	1.87	1.58	1.02	< 1.00
6	1.78	1.40	< 1.00	< 1.00
5	1.68	1.26	< 1.00	< 1.00
4	< 1.00	< 1.00	< 1.00	< 1.00
3	1.24	< 1.00	< 1.00	< 1.00
2	1.96	1.36	1.11	1.19
1	2.56	1.85	1.37	1.28

(b) Structures Designed to NZS 3101:1982

Table 5.2 Maximum Beam Curvature Ductility Demands for Segmental Structures

Types	Fundamental Periods (second)	
	Structures Designed to NZS 3101:1982	Structures Designed to NZS 3101:1995
Fixed Base Building	2.012	2.511
Base Isolated Building on a Rigid Base	2.087	2.548
Base Isolated Building on a Compliant Foundation	2.332	2.693
Segmental Building on a Rigid Base	2.261	2.700
Segmental Building on a Compliant Foundation	2.516	2.856

Table 5.3 Fundamental Periods for Different 12-storey Buildings

foundation decrease in stiffness much more than do those with a rigid base. Increase in the fundamental period due to foundation flexibility is also indicated in Refs. C6 and P9. From Table 5.3, the fundamental periods of structures designed to NZS 3101:1982 are different from those for structures designed to NZS 3101:1995 because the structures designed to the earlier code were required to be stiffer.

From Table 5.3, the period of the base isolated building is slightly lengthened when compared with the fixed base building. This is because the base isolation devices are only elastic at this stage. The yielding of the isolators will significantly increase the fundamental period of base isolated building.

5.4 Seismic Performances of Base Isolated Structures and Other Types of Structures

In this section, the structural response quantities such as lateral storey displacements, interstorey drifts, total acceleration, base shears and lateral storey shear envelopes obtained from the time history analyses are presented in order to show the typical performance of the base isolated structures. The structures were designed to NZS 3101:1995.

5.4.1 Lateral Storey Displacements and Interstorey Drifts

The lateral storey displacements and interstorey drifts are often used as a damage parameter to check whether or not the interstorey deflections of the building exceed acceptable limits during the seismic inelastic analyses. The interstorey deflections under seismic loading in many design codes [A6,B1,I1] must not exceed specified limits. Based on Section 2.5.4.5 (b) in NZS 4203:1992, the interstorey deflection shall not exceed 2.5% of the corresponding storey height when time history analyses are used. The 2.5% interstorey deflection limit seems a good starting point for the design of the long period (greater than 2 seconds) base isolated and segmental buildings.

When compared with the base isolated and segmental buildings with elasto-plastic isolation systems shown in Fig. 5.3, the fixed base building shows the greatest top floor displacement, though the dynamic characteristics do not change dramatically in that the overall natural period appears reasonably constant. Compared with the base floor

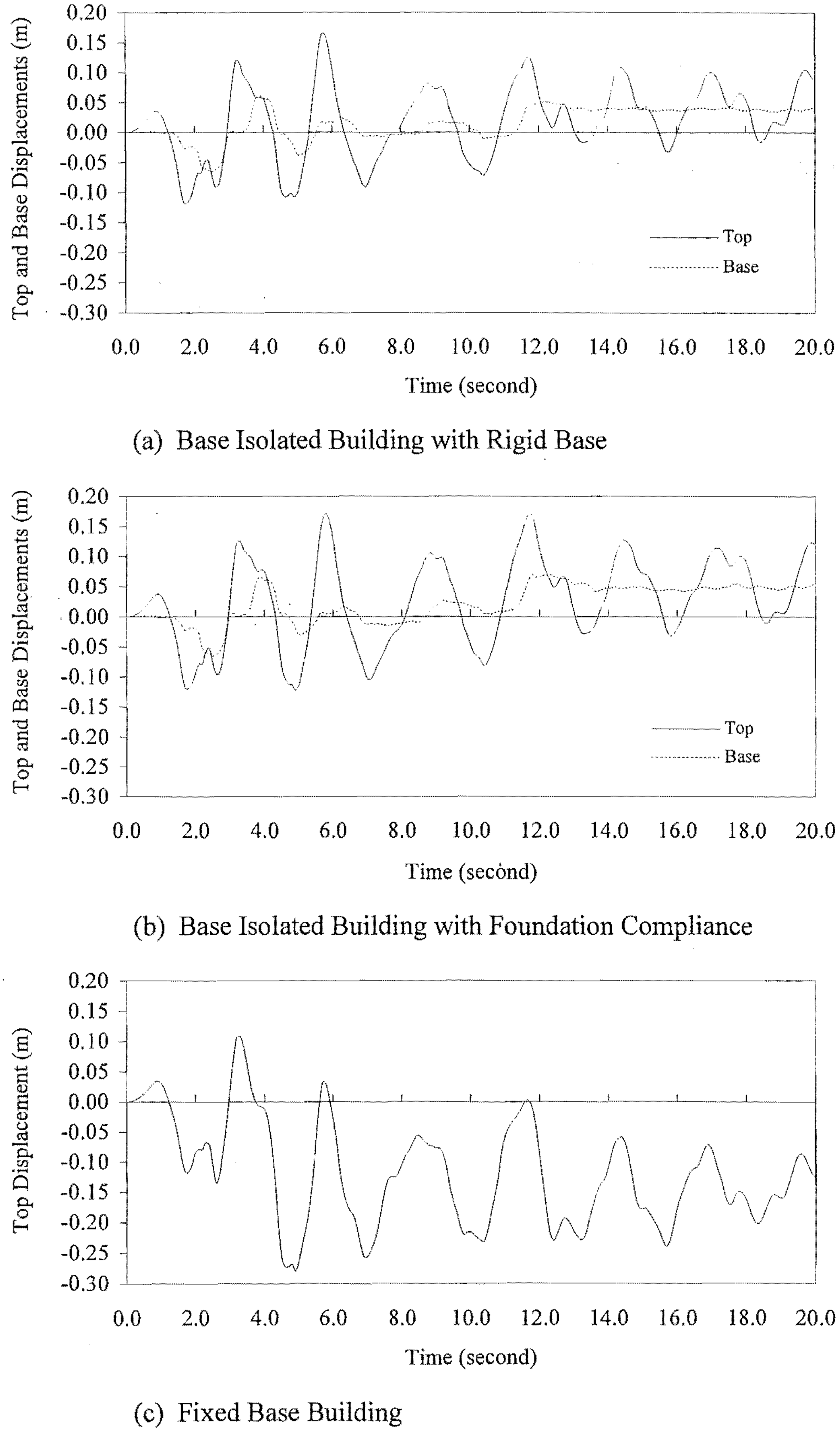
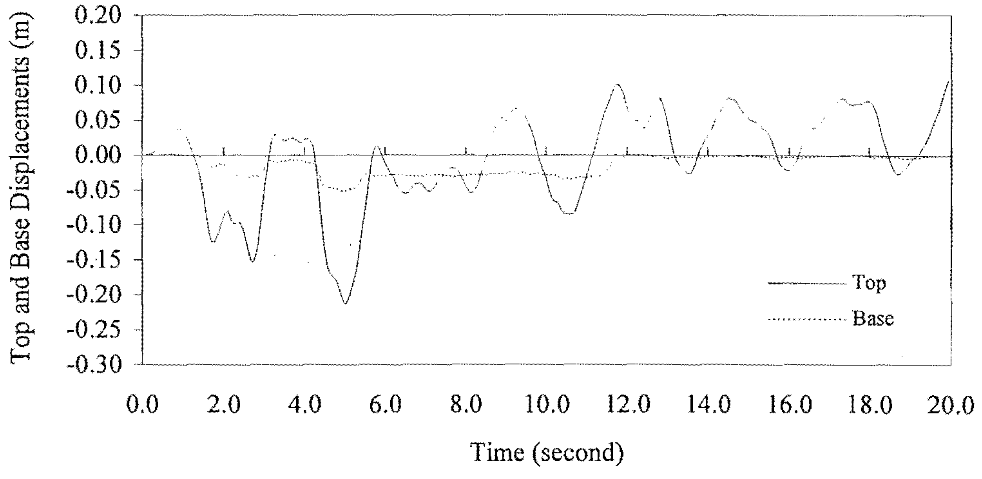
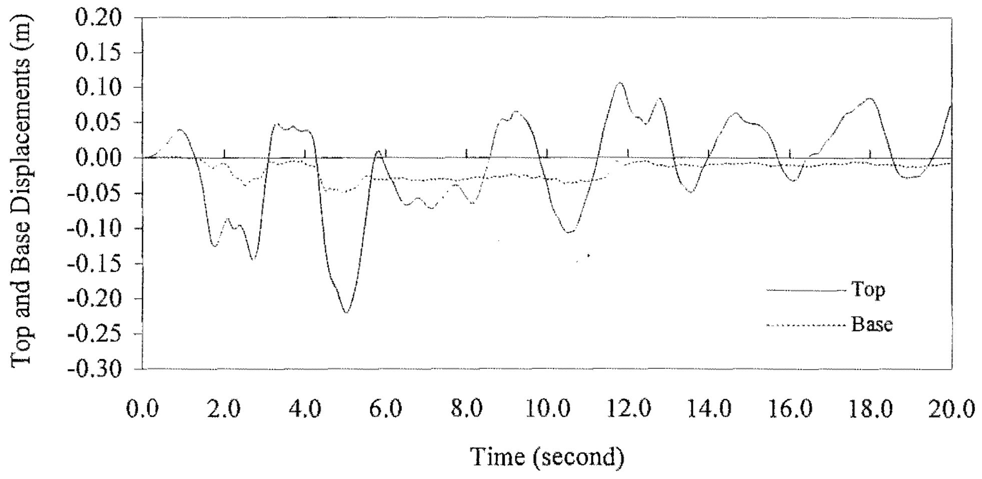


Fig. 5.3 The Response History of Top and Base Floor Displacements for Structures with Elasto-Plastic Isolation Systems



(d) Segmental Building with Rigid Base



(e) Segmental Building with Foundation Compliance

Fig. 5.3 ..... (continued)

displacement responses, the base isolated building with a compliant foundation shows the greatest displacement response. This is similar to the response obtained by the base isolated and segmental structures with bilinear isolation devices as shown in Fig. 5.4.

For the structures with elasto-plastic isolation systems shown in Fig. 5.5, the segmental buildings have smaller interstorey drifts than those observed in the base isolated buildings, and they give smaller interstorey drifts than the fixed base building. Similar responses are shown for the structures with bilinear isolation devices as given in Fig. 5.6.

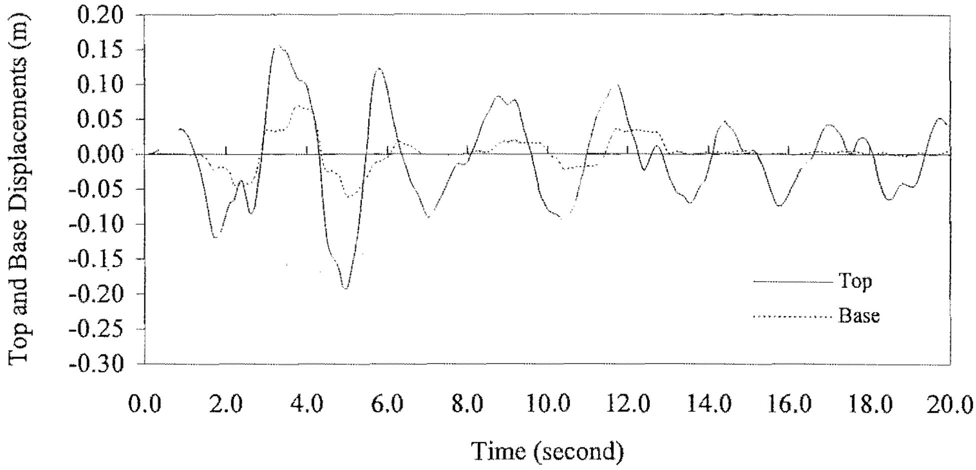
The interstorey drifts for the different structures are summarised Table 5.4. The base isolated and segmental structures with elasto-plastic and bilinear isolation systems have smaller interstorey drifts of less than 1.0%. In discussion of P-delta effects, Carr and Moss (1980) pointed out that if the interstorey drifts are less than 1.0%, the P-delta effect may be justifiably ignored. Similar results were obtained for structures designed to NZS 3101:1982 as shown in Table A.1 of Appendix A.

When compared with the fixed base building, the base isolated and segmental structures have much reduced interstorey drifts as shown in Fig. 5.7 and actually have much smaller displacements as well. This is true for both the elasto-plastic and bilinear isolation systems. Similar results are found for structures designed to NZS 3101:1982 as shown in Fig. A.1 of Appendix A.

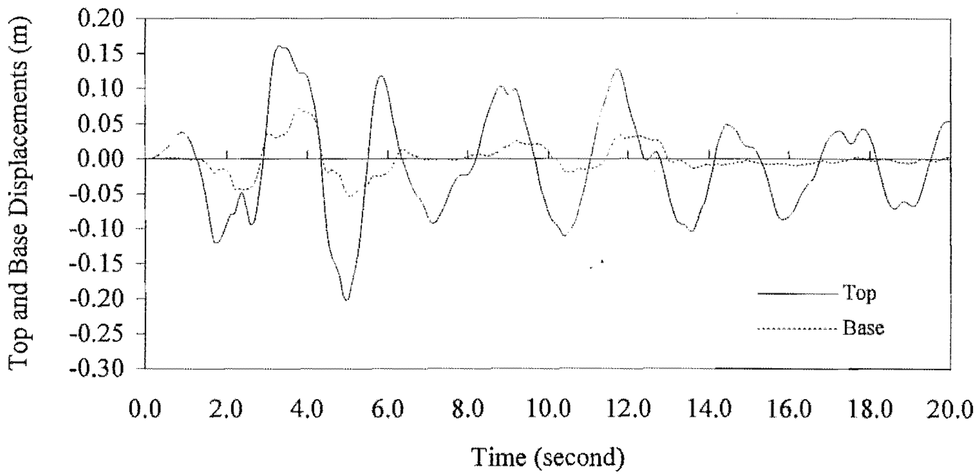
5.4.2 Total Acceleration

In general, seismic isolation limits the effects of the earthquake attack since a flexible base largely decouples the structure from the horizontal motion of the ground and the structural response accelerations are usually less than the ground acceleration.

From Fig. 5.8, the base isolated and segmental structures with elasto-plastic isolation systems at the base floor have smaller acceleration responses compared with those associated with the ground motions. This is also seen in the structures with bilinear isolation devices as shown in Fig. 5.9.

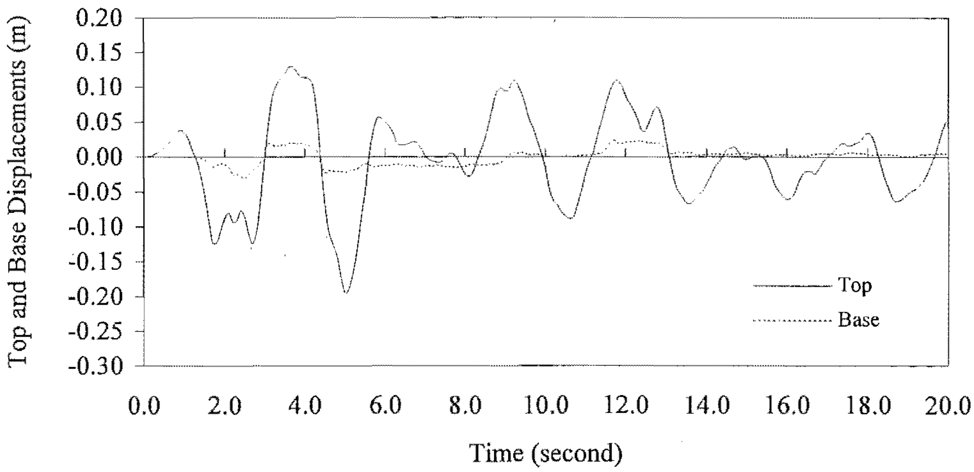


(a) Base Isolated Building with Rigid Base

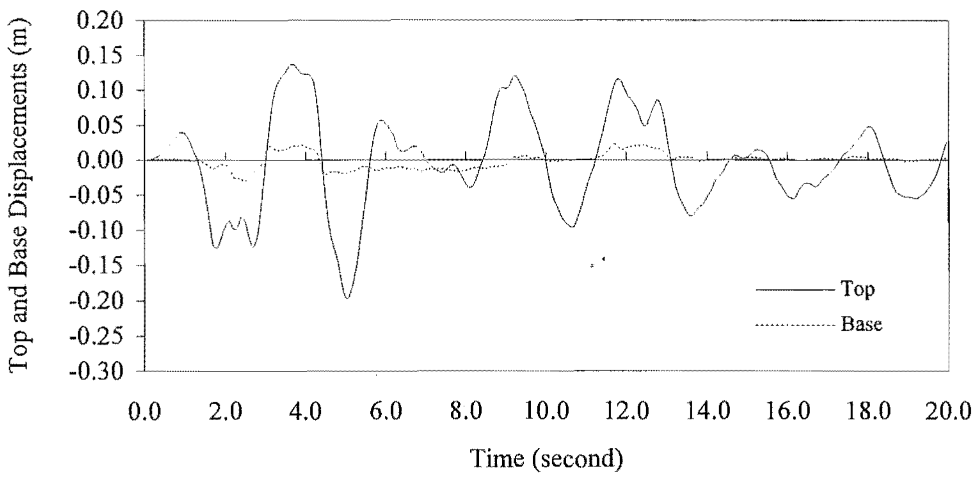


(b) Base Isolated Building with Foundation Compliance

Fig. 5.4 The Response History of Top and Base Floor Displacements for Structures with Bilinear Isolation Systems

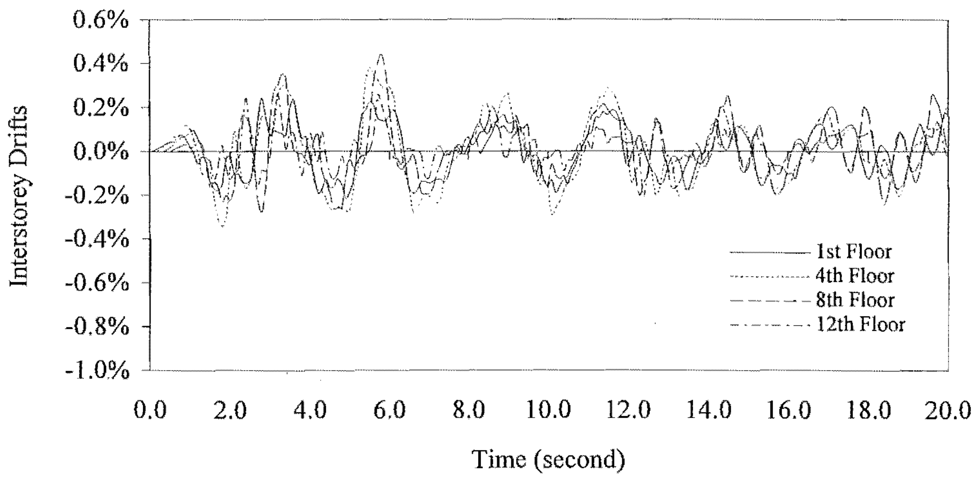


(c) Segmental Building with Rigid Base

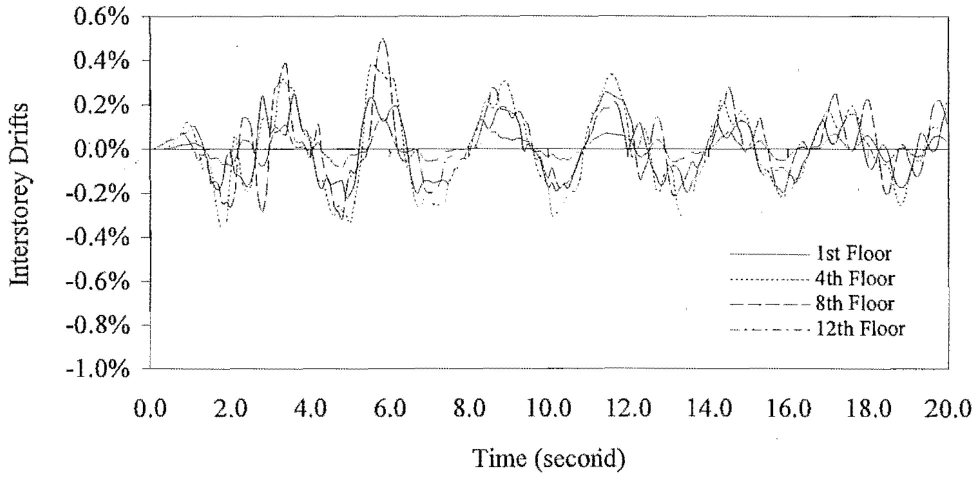


(d) Segmental Building with Foundation Compliance

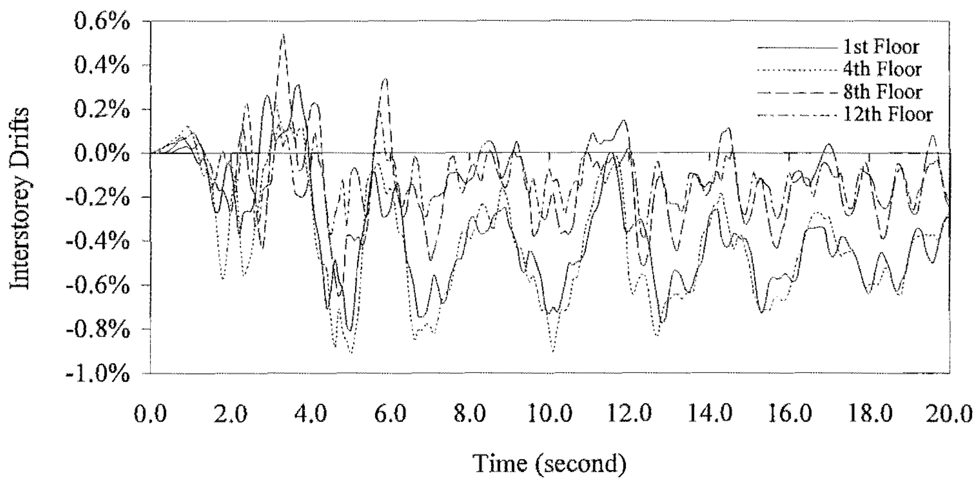
Fig. 5.4 ..... (continued)



(a) Base Isolated Building with Rigid Base

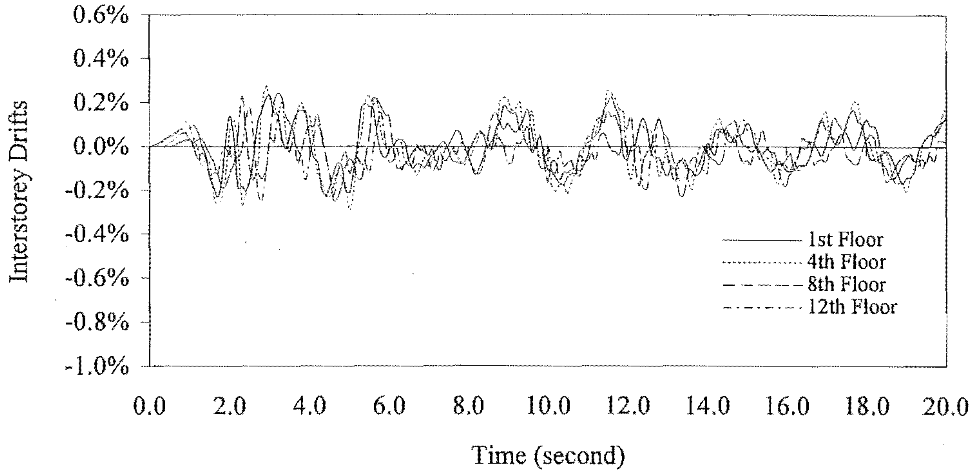


(b) Base Isolated Building with Foundation Compliance

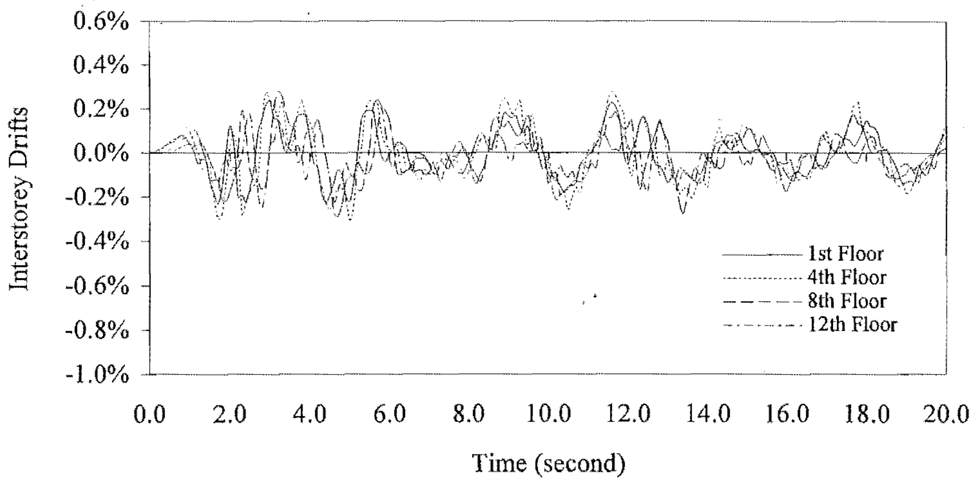


(c) Fixed Base Building

Fig. 5.5 The Response History of Interstorey Drifts for Structures with Elasto-Plastic Isolation Systems

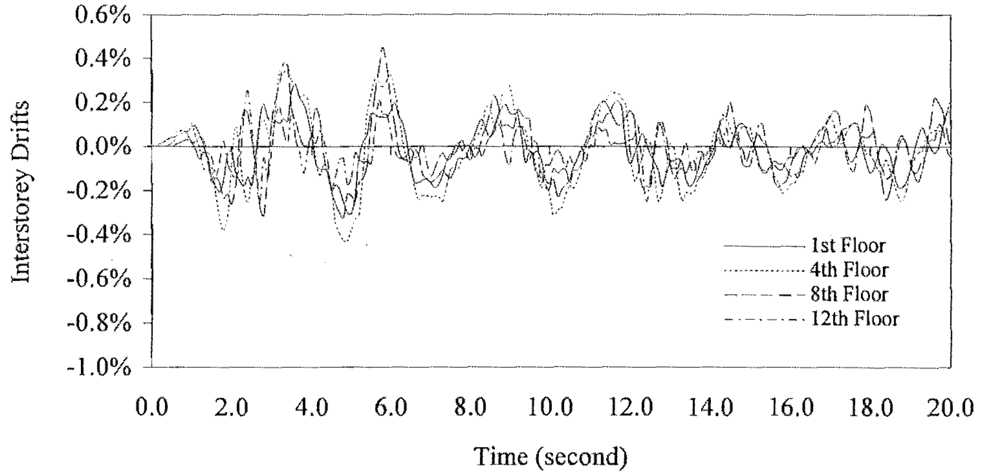


(d) Segmental Building with Rigid Base

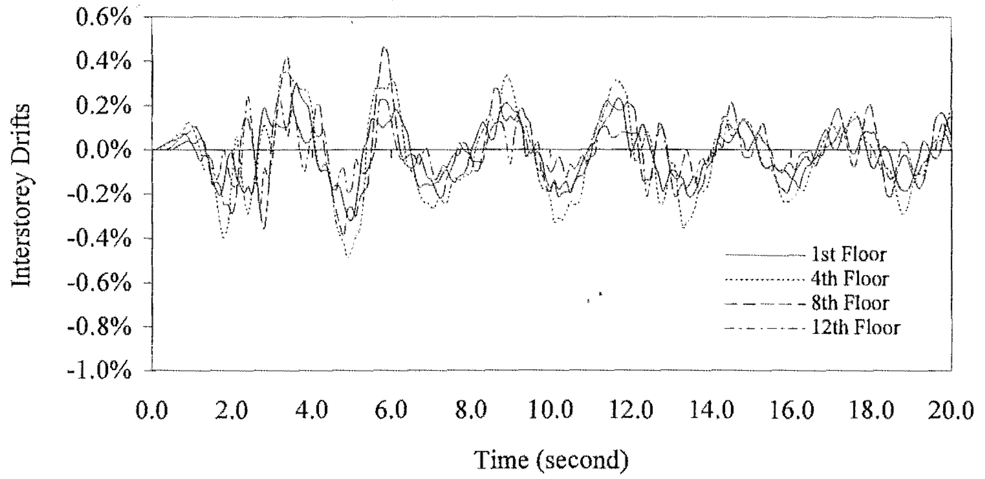


(e) Segmental Building with Foundation Compliance

Fig. 5.5 ..... (continued)

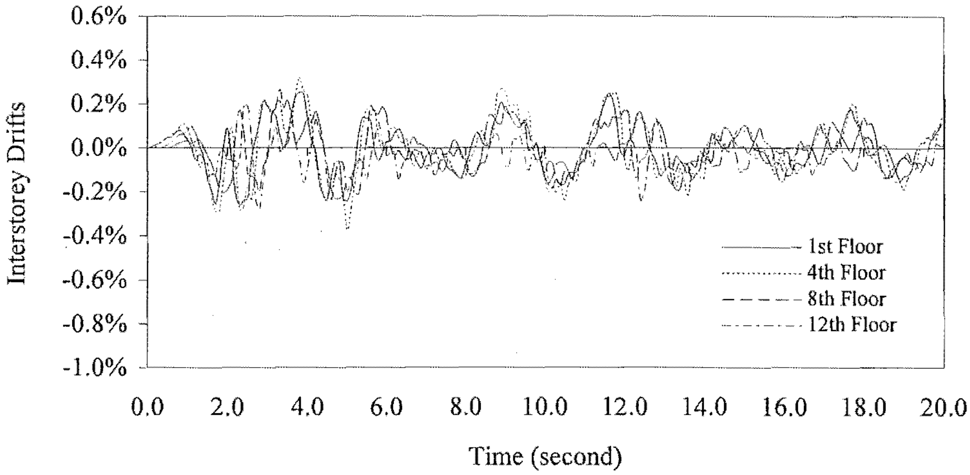


(a) Base Isolated Building with Rigid Base

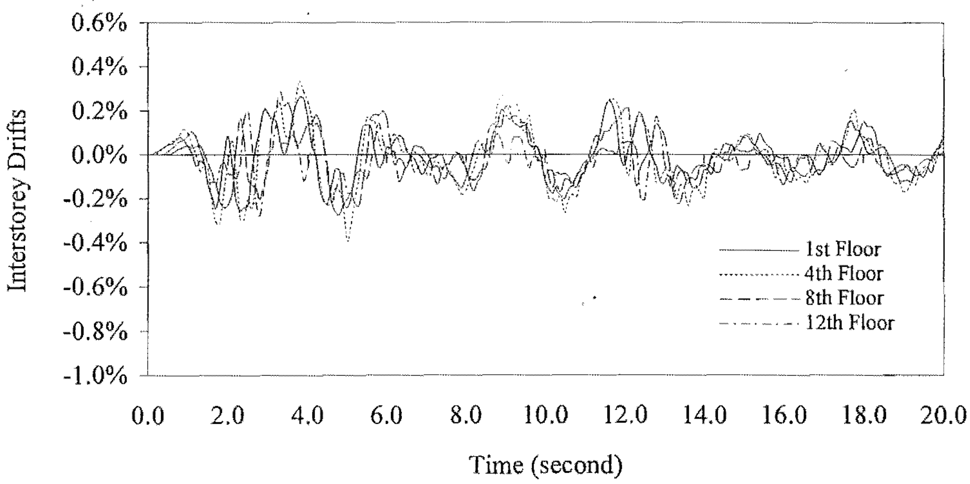


(b) Base Isolated Building with Foundation Compliance

Fig. 5.6 The Response History of Interstorey Drifts for Structures with Bilinear Isolation Systems



(c) Segmental Building with Rigid base



(d) Segmental Building with Foundation Compliance

Fig. 5.6 ..... (continued)

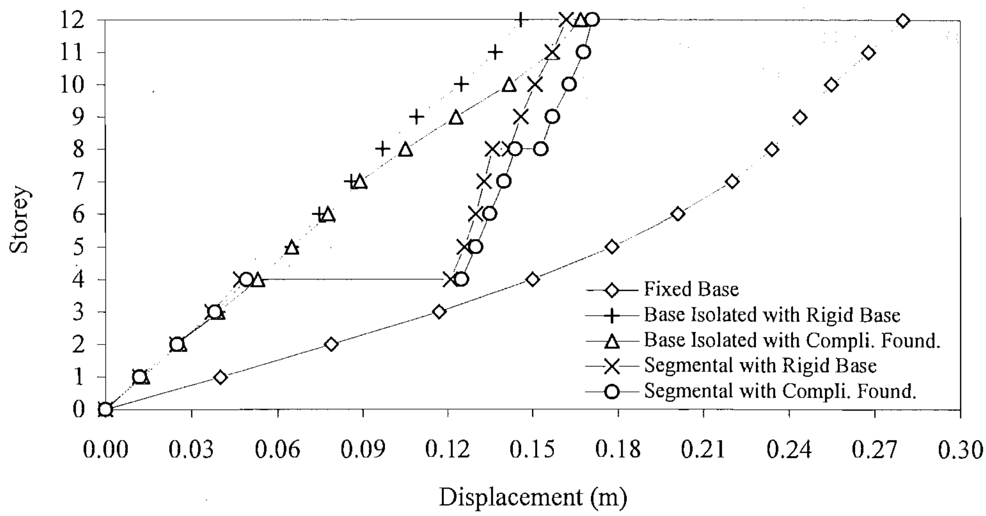
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.25%	0.27%	0.14%	0.08%
11	0.36%	0.33%	0.41%	0.16%	0.14%
10	0.30%	0.44%	0.52%	0.14%	0.16%
9	0.27%	0.33%	0.49%	0.11%	0.11%
8	0.38%	0.30%	0.44%	0.08%	0.11%
7	0.52%	0.30%	0.30%	0.08%	0.14%
6	0.63%	0.27%	0.36%	0.11%	0.14%
5	0.77%	0.33%	0.33%	0.14%	0.14%
4	0.90%	0.38%	0.38%	0.27%	0.30%
3	1.04%	0.38%	0.36%	0.33%	0.36%
2	1.07%	0.36%	0.36%	0.36%	0.36%
1	0.80%	0.24%	0.26%	0.24%	0.24%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

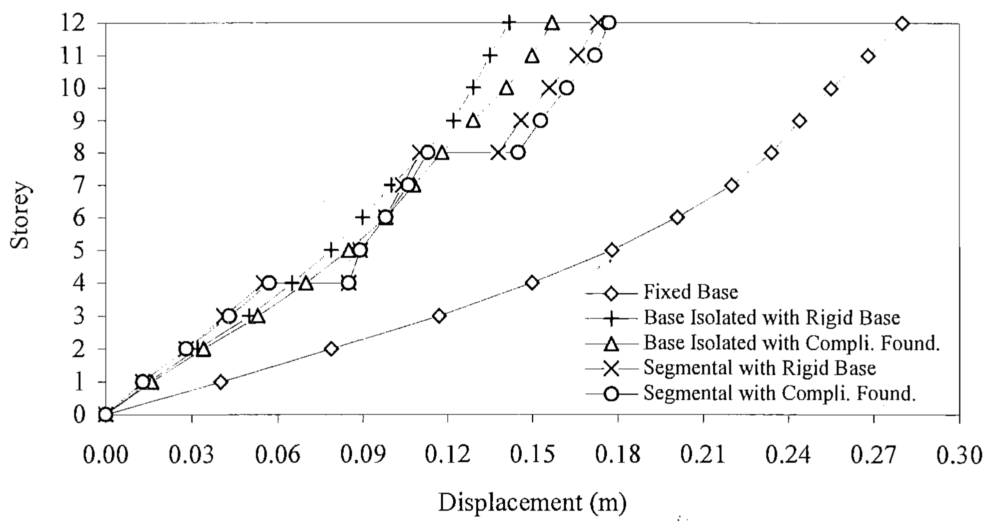
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.19%	0.19%	0.19%	0.14%
11	0.36%	0.16%	0.25%	0.27%	0.27%
10	0.30%	0.19%	0.33%	0.27%	0.25%
9	0.27%	0.27%	0.30%	0.22%	0.22%
8	0.38%	0.33%	0.27%	0.16%	0.19%
7	0.52%	0.27%	0.27%	0.16%	0.22%
6	0.63%	0.30%	0.36%	0.25%	0.25%
5	0.77%	0.38%	0.41%	0.11%	0.11%
4	0.90%	0.41%	0.47%	0.38%	0.38%
3	1.04%	0.49%	0.52%	0.36%	0.41%
2	1.07%	0.47%	0.49%	0.41%	0.41%
1	0.80%	0.30%	0.32%	0.26%	0.26%

(b) Structures Mounted on Bilinear Isolation Systems

Table 5.4 Interstorey Drifts for Different Types of Structures

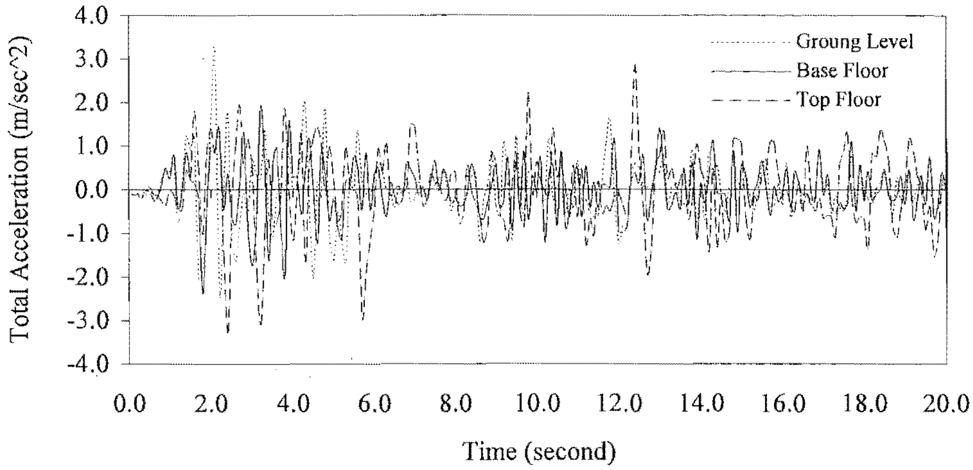


(a) Elasto-Plastic Model of the Isolation Systems

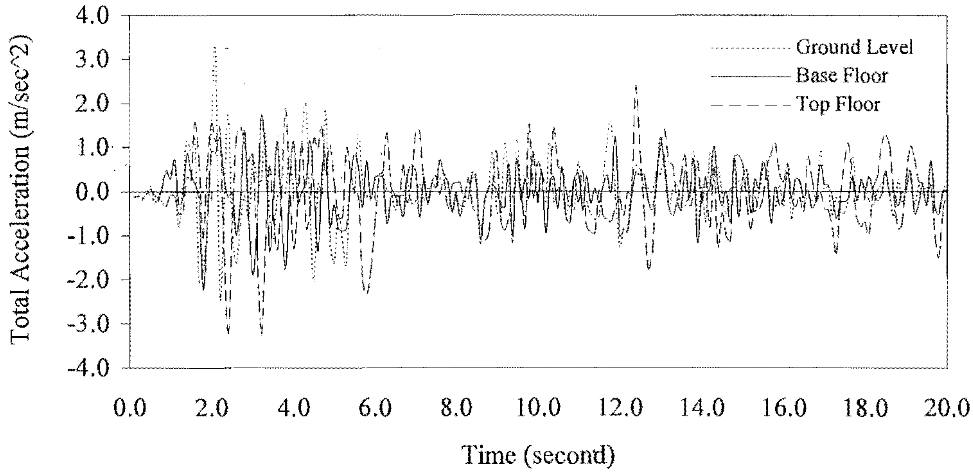


(b) Bilinear Model of the Isolation Systems

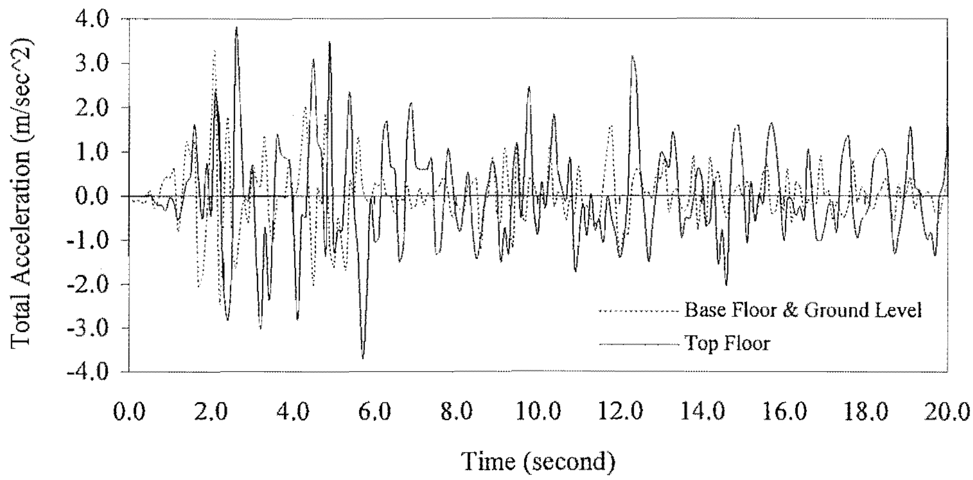
Fig. 5.7 Comparisons of Displacement with Storey for Fixed Base, Base Isolated and Segmental Buildings



(a) Base Isolated Building with Rigid Base

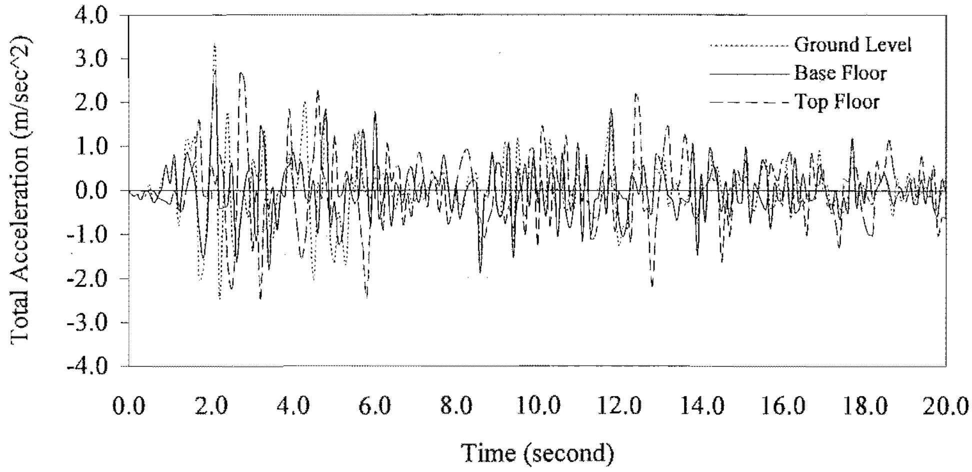


(b) Base Isolated Building with Foundation Compliance

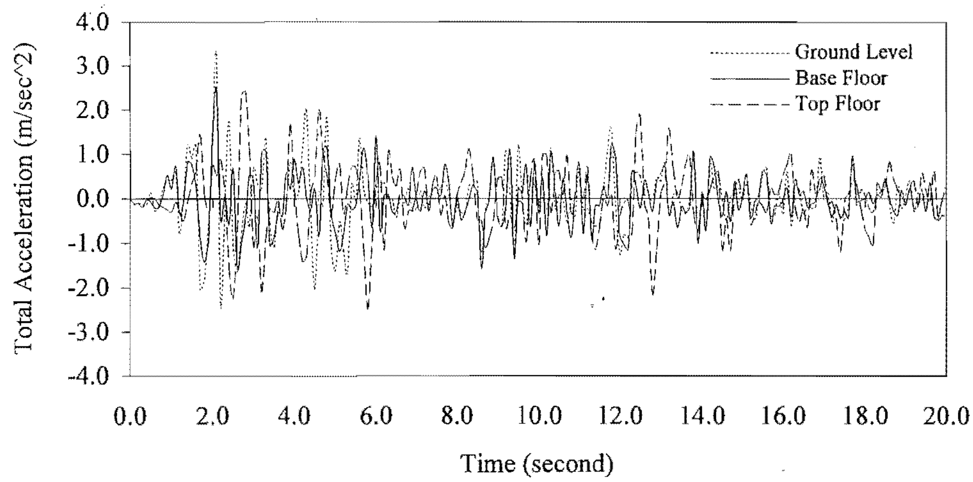


(c) Fixed Base Building

Fig. 5.8 The Response History of Total Accelerations for Structures with Elasto-Plastic Isolation Systems

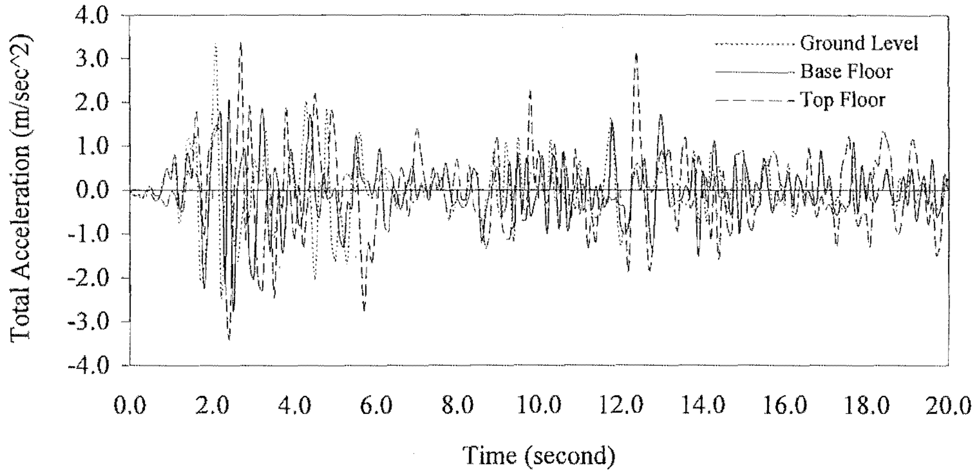


(d) Segmental Building with Rigid Base

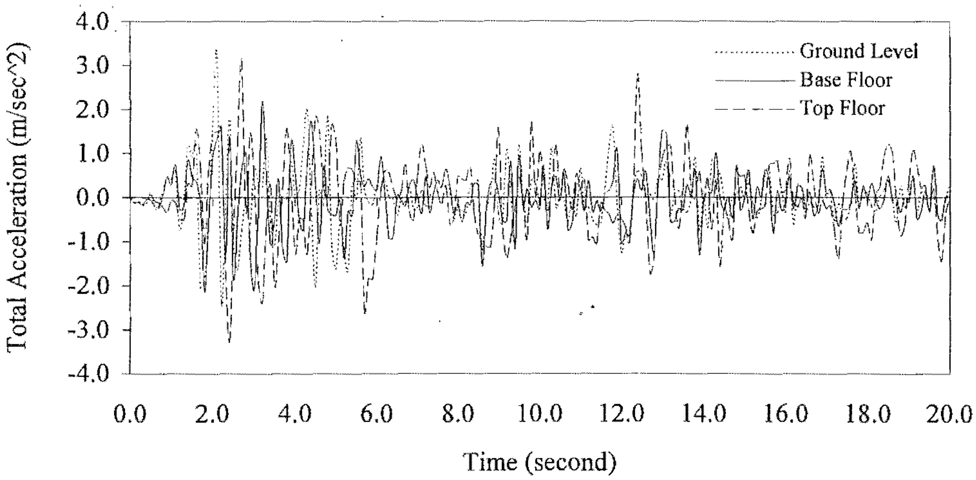


(e) Segmental Building with Foundation Compliance

Fig. 5.8 ..... (continued)

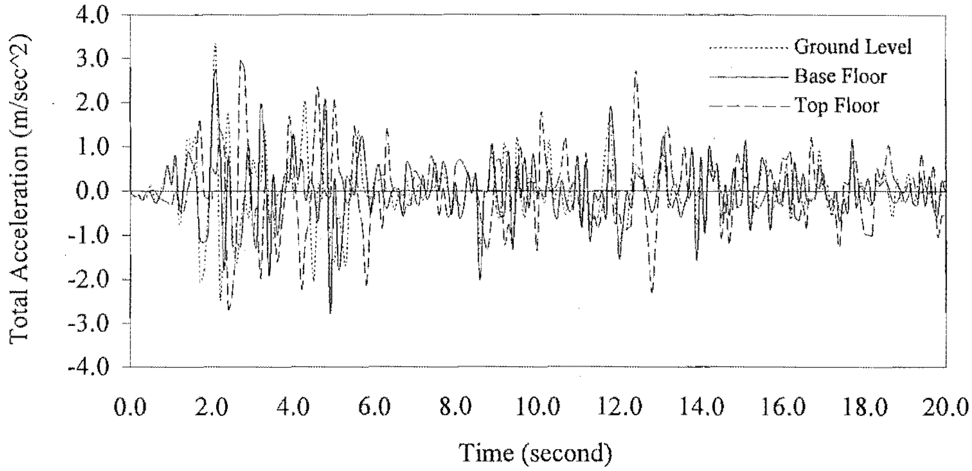


(a) Base Isolated Building with Rigid Base

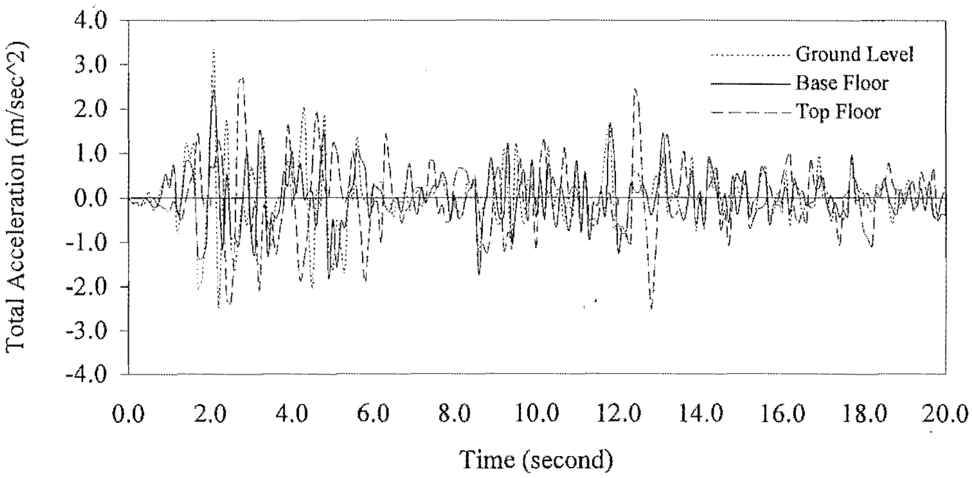


(b) Base Isolated Building with Foundation Compliance

Fig. 5.9 The Response History of Total Accelerations for Structures with Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation Compliance

Fig. 5.9 ..... (continued)

The response history plots of the structures with elasto-plastic isolators are shown in Fig. 5.8; the total acceleration (ground acceleration plus relative structural acceleration) at the base floor is greatly reduced by the base isolated building when compared with the fixed base building, though the segmental building has a slight smaller total acceleration than that for the fixed base building. When comparing the total acceleration responses, the segmental buildings show smaller total accelerations at the top floor than those observed in the base isolated buildings, and they give smaller total accelerations than the fixed base building. This is similar to results obtained using bilinear isolation systems as shown in Fig. 5.9.

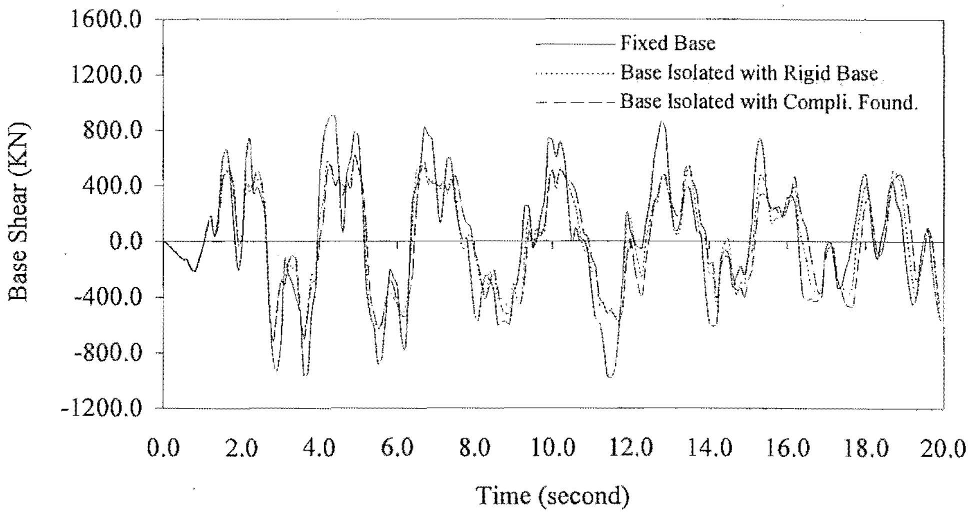
5.4.3 Base Shears and Lateral Storey Shear Envelopes

For design purposes, the base shear is normally regarded as a major response quantity by most loading codes. In this study, the base shear is used to describe the seismic performance for the base isolated structures. Besides the base shear, the other important parameter for a multistorey structure is its lateral storey shear envelope as indicated in Refs. A2, A3 and A4. Most of the loading codes relate this parameter to the equivalent static lateral force distribution over the height of the building.

For the buildings with elasto-plastic isolation systems shown in Fig. 5.10, the base shear responses for the base isolated and segmental structures are less than those for the fixed base buildings. Similar responses are observed in the structures using bilinear isolation devices as shown in Fig. 5.11.

Based on the time history analyses, the base shears of the base isolated and segmental buildings mounted on elasto-plastic and bilinear isolators shown in Table 5.5 are respectively 32%, 44%, 20% and 40% smaller than those of the fixed base buildings. For the structures designed to NZS 3101:1982, the base shears of the base isolated and segmental buildings with elasto-plastic and bilinear isolation devices are respectively 26%, 29%, 18% and 27% smaller than those of the fixed base buildings as shown in Table A.2 of Appendix A.

Also, compared with the equivalent static method of NZS 4203:1992 from Table 5.5, the base isolated and segmental buildings structures with elasto-plastic and bilinear isolation systems using the time history analyses have smaller base shears except for the fixed base



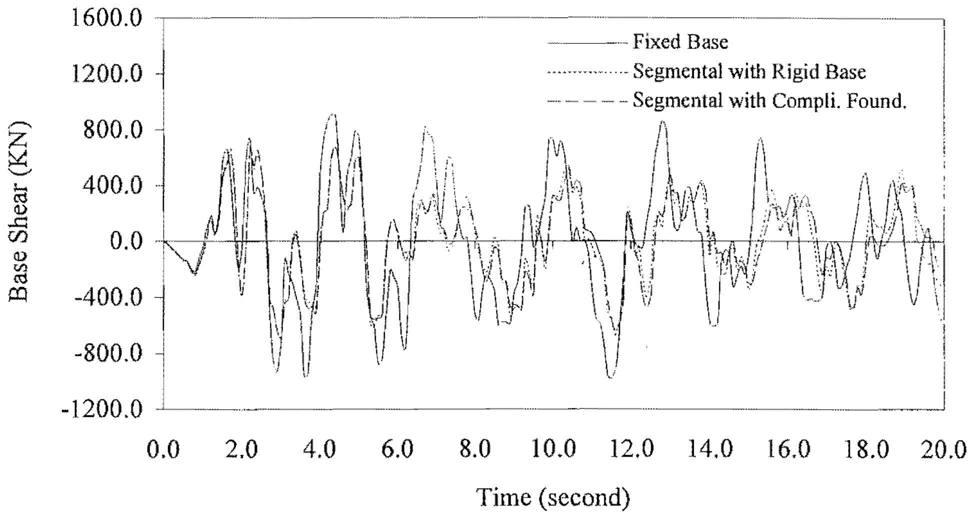
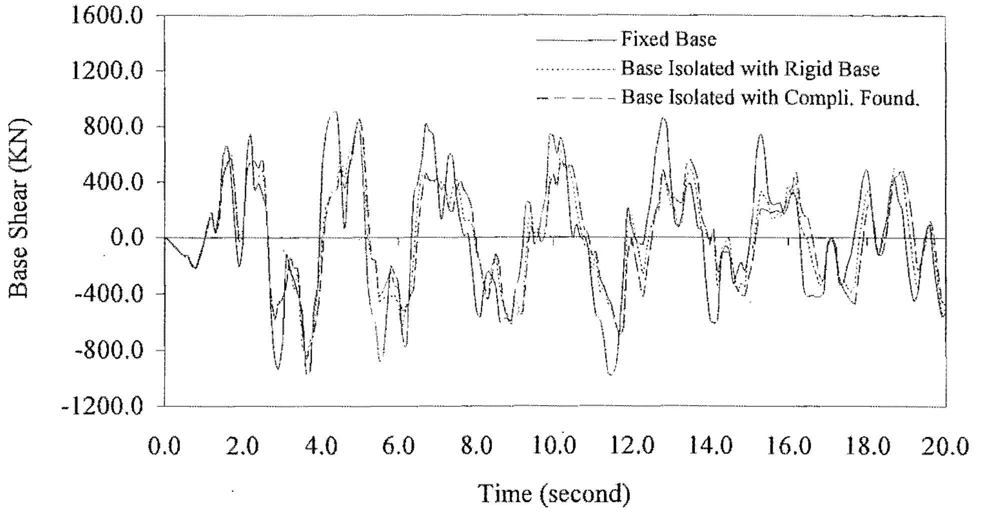


Fig. 5.10 The Response History of Base Shears for Structures with Elasto-Plastic Isolation Systems



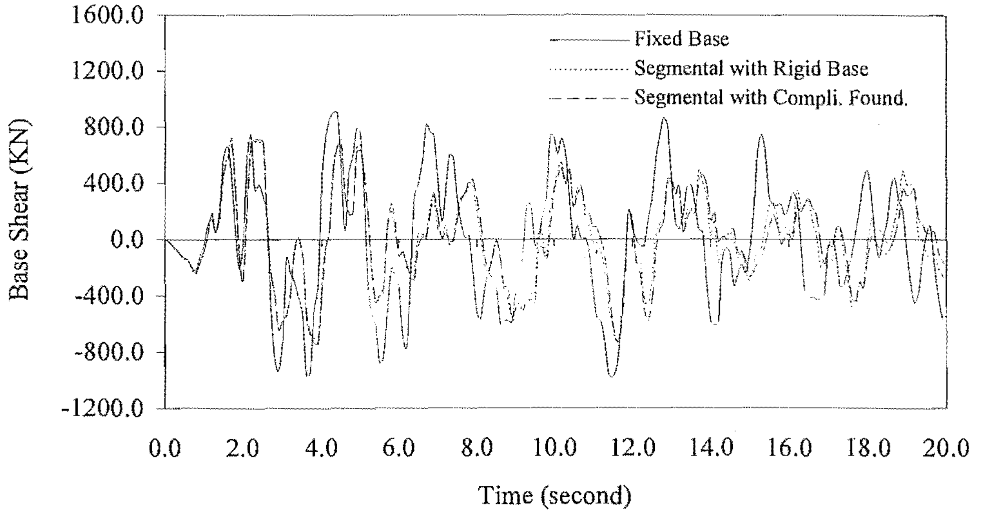


Fig. 5.11 The Response History of Base Shears for Structures with Bilinear Isolation Systems

Types	Base Shear / Total Weight of Structure		
	Time History Analysis		Equivalent Static Method of NZS 4203:1992
	Elasto-Plastic Model	Bilinear Model	
Fixed Base Building	0.0675		0.0480
Base Isolated Building on a Rigid Base	0.0453	0.0545	0.0480
Base Isolated Building on a Compliant Foundation	0.0457	0.0550	0.0480
Segmental Building on a Rigid Base	0.0373	0.0404	0.0480
Segmental Building on a Compliant Foundation	0.0375	0.0408	0.0480

Table 5.5 Normalised Base Shears for Different Types of Buildings

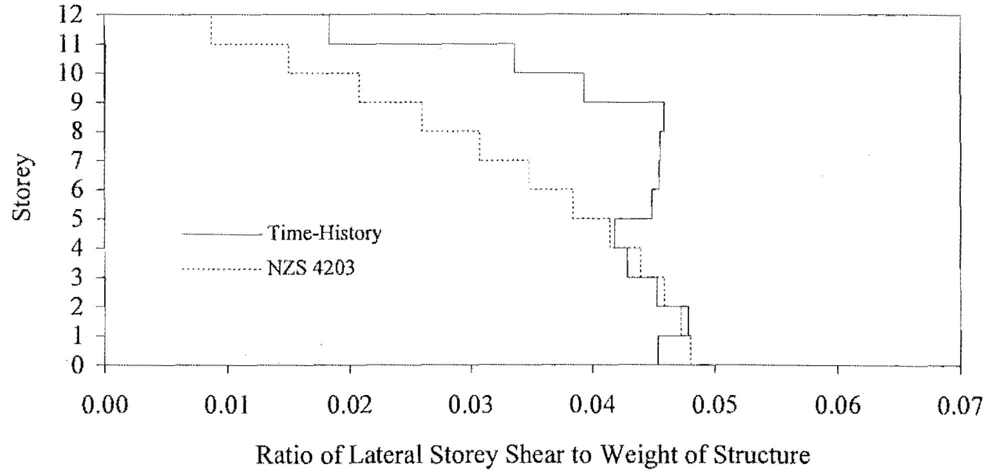
and base isolated building with bilinear isolation devices. Similar results are obtained for the structures designed to NZS 3101:1982 as shown in Table A.2 of Appendix A.

For the structures with elasto-plastic isolators, Fig. 5.12 shows the lateral storey shear envelopes of the base isolated, fixed base and segmental structures as obtained from the time history analyses and compares them with the equivalent static method of NZS 4203:1992. From the shear diagrams for the base isolated, fixed base and segmental structures, there are reductions of the storey shears over the height except in the middle storeys. This is also true for the buildings with bilinear isolation systems as shown in Fig. 5.13. This effect in the middle level storeys is produced by the contributions from the higher modes.

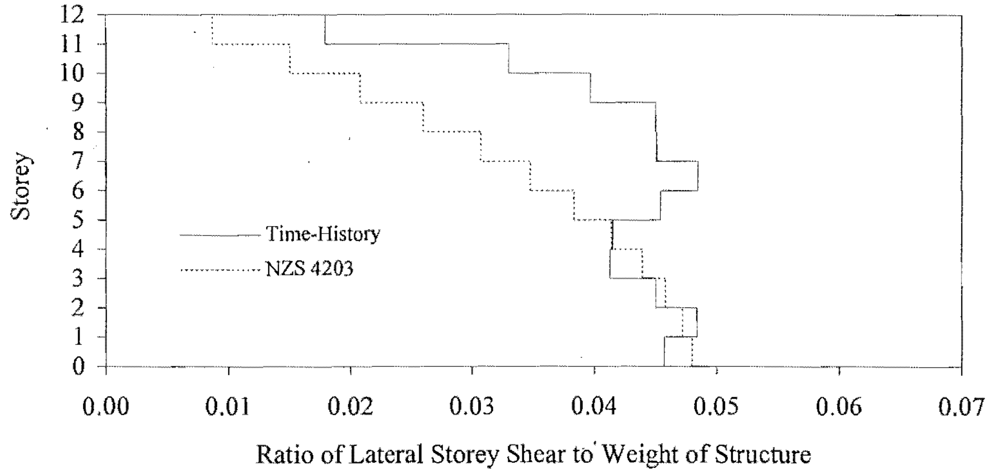
Compared with the time history analyses of Fig. 5.12, the equivalent static lateral force distribution recommended by NZS 4203:1992 gives a safety margin for the storey shears at the lower storeys and an underestimation of the storey shears at the upper storeys for both the base isolated and segmental buildings. For the fixed base building, there is an underestimation of the storey shears over the whole height of the structure.

For the structures with bilinear isolation systems, comparing the time history analyses with the equivalent static method shown in Fig. 5.13, the latter gives an underestimation of the storey shears over the height for the base isolated buildings, and a safety margin of the storey shears at the lower storeys and an underestimation of the storey shears at the upper storeys for the segmental buildings.

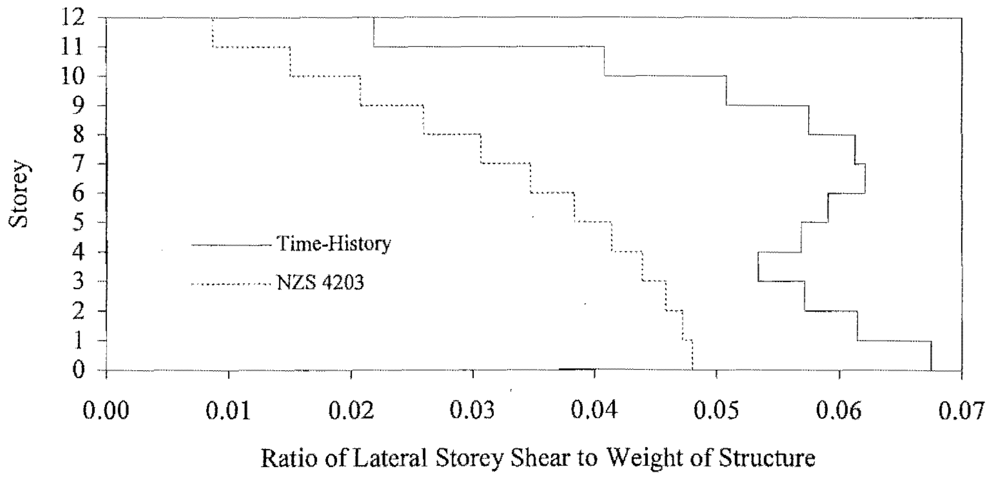
For the structures with elasto-plastic isolators, the lateral storey shear envelopes for the base isolated, fixed base and segmental buildings designed to NZS 3101:1982 are shown in Fig. A.2 of Appendix A. Compared with time history analyses, the equivalent static lateral force distribution of NZS 4203:1992 gives a smaller estimation of the storey shears for the base isolated and fixed base buildings, and a safety margin of the storey shears at the lower storeys and an underestimation of the storey shears at the upper storeys for the segmental buildings. This is also true for the structures with bilinear isolation systems as shown in Fig. A.3 of Appendix A.



(a) Base Isolated Building with Rigid base

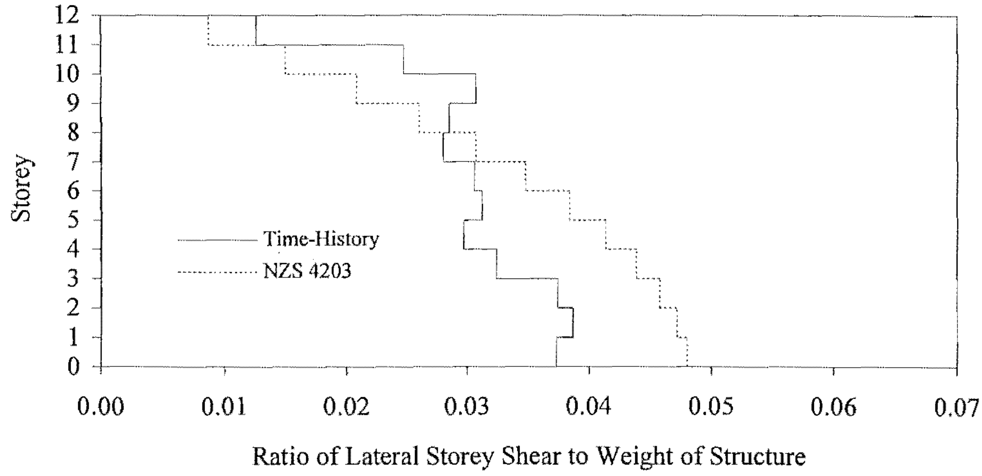


(b) Base Isolated Building with Foundation Compliance

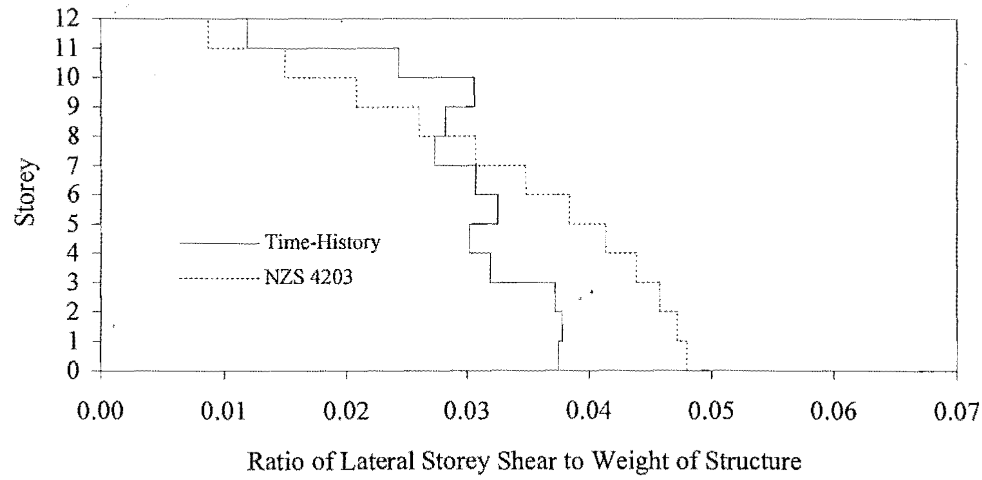


(c) Fixed Base Building

Fig. 5.12 Lateral Storey Shear Envelopes for Structures with Elasto-Plastic Isolation Systems

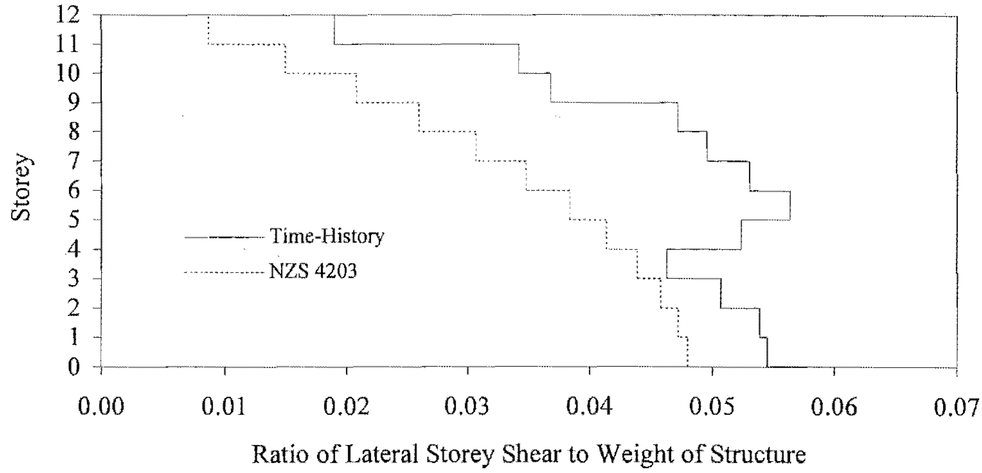


(d) Segmental Building with Rigid Base

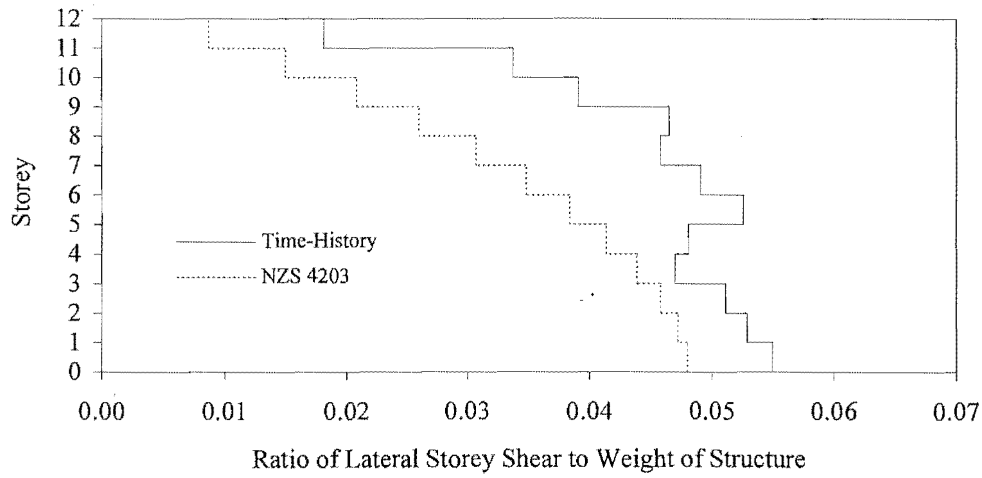


(e) Segmental Building with Foundation Compliance

Fig. 5.12 ..... (continued)

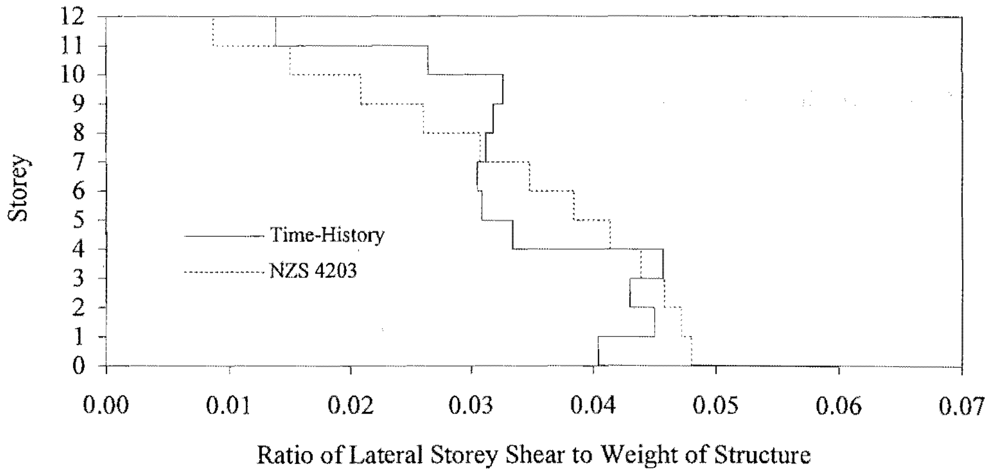


(a) Base Isolated Building with Rigid Base

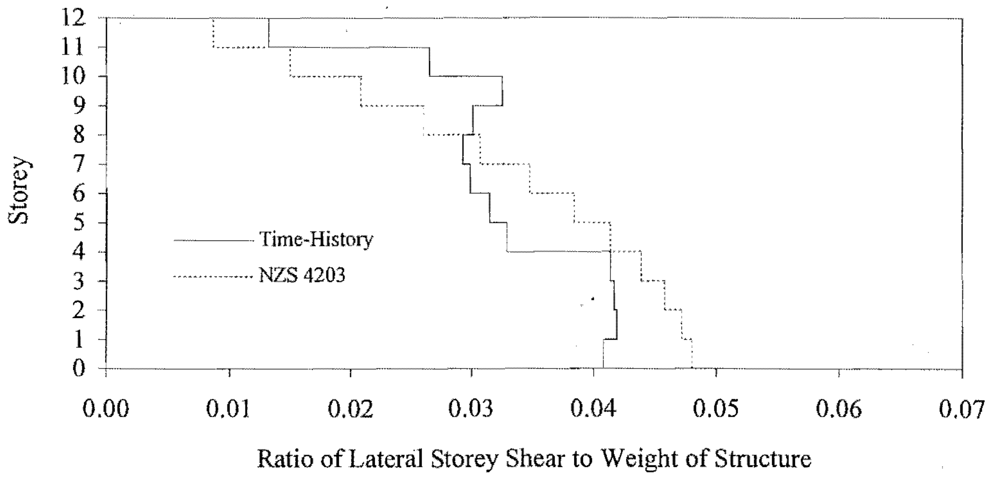


(b) Base Isolated Building with Foundation Compliance

Fig. 5.13 Lateral Storey Shear Envelopes for Structures with Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation Compliance

Fig. 5.13 ..... (continued)

The purpose of this study is to understand the difference in the lateral storey force distributions for the base isolated and segmental buildings during the time history analyses when compared with the equivalent static lateral force distribution of NZS 4203:1992.

5.5 Curvature Ductility Demands of Beams and Columns

As the structures were designed according to the capacity design method [P2,P5], very few column hinges appeared in the time history analyses except at the column bases. Thus, the member ductility factors mentioned in this study are limited to the beam hinges at the beam ends and column hinges at the base of the columns respectively. In the strong column and weak beam type of failure mechanism, the member ductility demand is the most common damage parameter for calculating the degree of damage in the members.

The member damage index depends strongly on the ultimate curvature of the members and it is very important to accurately evaluate the ultimate curvature for the members. However, it is very difficult to define the ultimate state of the members. Carr and Tabuchi (1993) [C2] assumed an ultimate curvature ductility factor of 30 when using the damage index developed by Park and Ang [P3,P4].

Priestley et al (1981) reported that tests of spirally-confined concrete columns with a relatively low axial load ratio show that the range of the maximum available curvature ductility factors are from 16 to 25. Similar tests have been conducted by Park et al (1982) with the maximum curvature ductilities varying between 20 and 21. Zahn et al (1986) and Watson et al (1994) showed that if the axial load ratio is less than 0.15, a curvature ductility even greater than 30 may be obtained from a proposed curvature ductility design chart. Based on the laboratory tests mentioned above, maximum curvature ductility factors of 20 and 30 for columns and beams are used in this study. In this section, the 12-storey reinforced concrete moment-resistant frame models were used as discussed in Sections 3.4.3 and were designed to NZS 3101:1995.

For the elasto-plastic isolation systems as shown in Table 5.6 (a), the curvature ductility demands at the internal and external column bases are 2.16 and 2.28 for the fixed base building, less than 1.0 for the base isolated and segmental buildings with either a rigid base or a compliant foundation. This is similar to results obtained for the base isolated and segmental structures with bilinear isolation systems as shown in Table 5.6 (b). The curvature ductility demand (less than 1.0) in the table implies that the member remains elastic, i.e. no plastic hinge occurs.

As shown in Table 5.6 (a), the maximum beam curvature ductility demand is 7.01 for the fixed base building. The maximum beam curvature ductility demands are 1.99 and 2.31 for the base isolated buildings with a rigid base and a compliant foundation, and 2.30 and 2.52 for the segmental buildings with a rigid base and a compliant foundation, respectively. These show that the maximum curvature ductility demands for the base isolated and segmental buildings with a rigid base and a compliant foundation using elasto-plastic isolation systems are 72%, 67%, 67% and 64% smaller than those for the fixed base buildings, respectively. Table 5.6 (b) shows very similar results for the base isolated and segmental structures using bilinear isolation systems.

For the structures designed to NZS 3101:1982 shown in Table A.3 of Appendix A, the curvature ductility demands at the internal and external column bases for the base isolated and segmental buildings using elasto-plastic and bilinear isolation systems are much smaller than those for the fixed base buildings. For the maximum beam curvature ductility demands, the base isolated and segmental buildings with elasto-plastic and bilinear isolation devices are respectively 75%, 74%, 73% and 72% smaller than those for the fixed base buildings.

5.6 Summary and Conclusion

A series of time history analyses have been carried out to investigate in detail seismic responses of a wide range of base isolated multistorey structures with elasto-plastic and bilinear isolation devices and comparisons of other types of structures such as fixed base and segmental buildings, under the N-S component of El Centro 1940 earthquake.

From Section 5.2, it can be seen that the post-yield stiffness of a bilinear isolation system for a 12-storey multistorey base isolated structure is significantly reduced when compared with that for the 4-storey building proposed by Andriono (1990) [A2], based on similar initial stiffness and yield strength of a base isolation system. For base isolated structures with more flexible superstructure, such as when fundamental period is over 2.0 seconds, the maximum storey shears may not always occur at the base. This is because of the contributions from the higher modes.

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	2.12	< 1.00	< 1.00	< 1.00	< 1.00
	11	3.62	1.34	1.52	1.96	2.09
	10	4.98	1.44	1.73	2.28	2.38
	9	5.34	1.99	2.31	2.30	2.52
	8	4.62	1.31	1.71	< 1.00	< 1.00
	7	5.46	1.08	1.47	< 1.00	1.27
	6	6.37	< 1.00	< 1.00	< 1.00	< 1.00
	5	6.15	< 1.00	< 1.00	< 1.00	< 1.00
	4	6.56	< 1.00	< 1.00	< 1.00	< 1.00
	3	6.95	< 1.00	< 1.00	< 1.00	< 1.00
	2	7.01	< 1.00	< 1.00	< 1.00	1.03
	1	6.98	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	2.28	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.16	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.26	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	2.12	< 1.00	< 1.00	< 1.00	1.13
	11	3.62	1.77	1.90	2.18	2.43
	10	4.98	1.91	1.78	2.52	2.54
	9	5.34	1.89	2.22	2.51	2.62
	8	4.62	1.25	1.59	< 1.00	< 1.00
	7	5.46	1.35	1.66	< 1.00	1.17
	6	6.37	1.22	1.47	< 1.00	< 1.00
	5	6.15	1.42	1.44	< 1.00	< 1.00
	4	6.56	1.14	1.33	< 1.00	< 1.00
	3	6.95	1.29	1.70	< 1.00	1.43
	2	7.01	1.49	1.89	< 1.00	1.29
	1	6.98	1.31	1.54	< 1.00	1.05
Column Bases	L.Ext.*	2.28	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.16	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.26	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:
* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 5.6 Maximum Curvature Ductility Demands for Different Structures

In Section 5.3, the fundamental periods of structures were given for fixed base, base isolated and segmental buildings based on two design standards, NZS 3101:1982 and NZS 3101:1995. From the results obtained, the fundamental periods of buildings designed according to the former code are smaller than those based on the latter one because the former structures are required to be stiffer.

It was shown in Section 5.4 that there are benefits in implementing a base isolation system by comparing the performance of the base isolated, segmental and fixed base multistorey buildings. With the inclusion of the isolation devices, the base isolated and segmental buildings with elasto-plastic and bilinear isolation systems have greatly reduced interstorey drifts and base shears compared with those for the fixed base buildings. The much smaller interstorey drifts avoid the early occurrence of non-structural damage during moderate earthquakes.

From the inelastic time history analyses, the base isolated and segmental buildings with elasto-plastic and bilinear isolation systems show that there is a reduction in the storey shears over the height except at the middle level storeys. This is because as the superstructure becomes more flexible, the higher modes make more significant contributions especially in the middle height section of the building.

As shown in Sections 5.4, the segmental building possesses the ability to decouple the building from the harmful horizontal earthquake ground motions in a manner similar to that of the base isolated building. While keeping the ratio of yielding force of isolation system to weight of structure low, a segmental building significantly reduces the base displacement response compared with that for a base isolated building.

The curvature ductility demands at beam ends and column bases for the fixed base, base isolated and segmental buildings under the El Centro 1940 N-S earthquake are presented in Section 5.5. The base isolated and segmental buildings with elasto-plastic and bilinear isolation systems have dramatically reduced maximum beam curvature ductility demands when compared with those for the fixed base buildings. This is true for the structures designed to NZS 3101:1995 and NZS 3101:1982 respectively. Due to the reduced ductility demands, the structural members of the base isolated and segmental buildings may not need to be designed to comply with fully ductile design requirements.

As can be seen from Sections 5.4 and 5.5, the effect of foundation compliance on the protection provided by the base isolation system was investigated to compare with the case of a rigid base foundation. The lateral displacements in the upper parts of the base isolated and segmental buildings with foundation compliance show a greater difference when compared with the same buildings with a rigid base. This is because of the contributions from a rocking mode in the structures with foundation compliance. It can be noted that the increases in displacements of the structure with compliant foundation are almost linear with height above the foundation. This shows that the predominant effect of the compliance is a rocking mode. Therefore, the effects of rocking need to be considered in the design of long period base isolated and segmental buildings with foundation compliance.

CHAPTER 6

THE SEISMIC RESPONSES OF STRUCTURES WITH ADDED DAMPING DEVICES SUBJECTED TO THE 1940 EL CENTRO N-S EARTHQUAKE

6.1 Introduction

The seismic responses of the base isolated building with elasto-plastic and bilinear isolation systems and comparisons of the fixed base and segmental buildings when subjected to the El Centro 1940 N-S earthquake were presented in the previous chapter. Based on the results obtained, the base isolation device has significantly attenuated the transmitted ground motion energy into the structure. Besides the base isolation systems, it has been suggested that the added damping devices installed in a structure may also be suitable for improving the seismic resistance of buildings [A1,S7]. From the examination of any response spectrum it can be seen that the provision of additional damping in a structure will reduce the magnitude of both the acceleration and displacement responses. Therefore, an approach aimed at evaluating the suitability of the added damping systems for seismic design will be undertaken in this chapter.

The primary purpose of this chapter is the use of velocity-dependent damping systems as energy absorbing devices to investigate whether they can be effective in reducing structural response to seismic excitation when installed in a building structure. The additional damping devices as shown in Fig. 6.1 are installed at the interior columns of each floor of the uniform superstructure models mentioned in Sections 3.4.3. The horizontal damping forces are proportional to the difference in the horizontal velocities of the two adjoining floors. An approach based on nonlinear dynamic analyses will be developed according to the use of simplified energy calculations to estimate the required hysteretic damping needed to obtain the desired equivalent viscous damping.

With the added viscous damping devices installed in the structure, the natural frequency of the building is not significantly altered. Thus, the very minor reduction in the period of the structure is not considered.

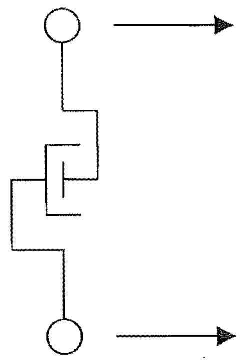


Fig. 6.1 Additional Damping Device

6.2 Additional Equivalent Viscous Damping Used in This Study

6.2.1 General

In order to investigate the suitability of added damping devices for seismic design applications, the effective fundamental period of the structure must firstly be evaluated and then the additional equivalent viscous damping coefficient and the additional damping ratio from the time history analyses can be computed. In this study, the effective fundamental period in a structure will be evaluated from the time history plots suggested by Turkington et al (1987) [T3,T4] and compared with that from the free-vibration modal analyses.

Before achieving the effective fundamental period of the structure, a loading time history was set up based on the equivalent static method of NZS 4203:1992 [S8] and then using the computer program *Ruaumoko* [C1] for conducting nonlinear dynamic analyses. This procedure of obtaining the additional equivalent viscous damping in a structure will be described below.

6.2.2 Loading Time History

Loads can be applied to the structures as one pattern which is multiplied by its loading time history. In this study, it was assumed that the loading time history is subjected to a harmonically varying load of sine-wave form having an amplitude and circular frequency [C7,C8]. Based on the equivalent static method of NZS 4203:1992, the maximum harmonically applied load is determined according to the horizontal seismic shear force V acting at the base of the structure in the direction being considered which can be calculated from

$$V = C\ W_t \tag{6.1}$$

in which

$$C = C_h\ (T_1\ ,\ \mu)\ S_p\ R\ Z\ L_u \tag{6.2}$$

where C and W_t are the lateral force coefficient and the total seismic weight of the structure respectively; $C_h(T_1, \mu)$ is basic seismic hazard acceleration coefficient based on intermediate soil sites and T_1, μ are the fundamental period of vibration for the direction being considered and the structural ductility respectively; S_p, R, Z, L_u are the structural performance factor, the risk factor, the zone factor and the limit state factor respectively.

In this study, it is assumed that the value of T_1, the fundamental period of the unisolated structure varies from 0.2 to 2.0 seconds, and the structural ductility of μ is determined according to comparison of base shears of the structures from earthquake-induced and wind-induced loads shown in Table 4.1 and Fig. 6.2. For selection of the structural ductility, it is assumed that the base isolation devices used in the buildings are only available when base shear attained under the earthquake-induced load is greater than that achieved by wind-induced load. Based on the provisions of NZS 4203:1992, the structural ductilities used are respectively 3.0 and 4.0 for the structures designed to NZS 3101:1995 [S9] and NZS 3101:1982 [N1].

6.2.3 Determination of Effective Period

As demonstrated by Andriono (1990) [A2,A3], it is known that the effective fundamental periods of the base isolated structures are accompanied by an increase of damping from those of the fixed base structures. Therefore, it is worthwhile to compare the effect of the stiffness of the superstructure on the seismic response of the fixed base and base isolated structures. For this purpose, a series of 12-storey frame structures deforming in a shear-like manner are considered. The fundamental period of these structures on a fixed base $T_{1\,(UI)}$ varies from 0.2 to 2.0 seconds.

The base isolation system on which the structure is mounted has an initial stiffness k_o of 10.0 W/m, a post yield stiffness αk_o of 0 and 0.4 W/m for the elasto-plastic and bilinear models

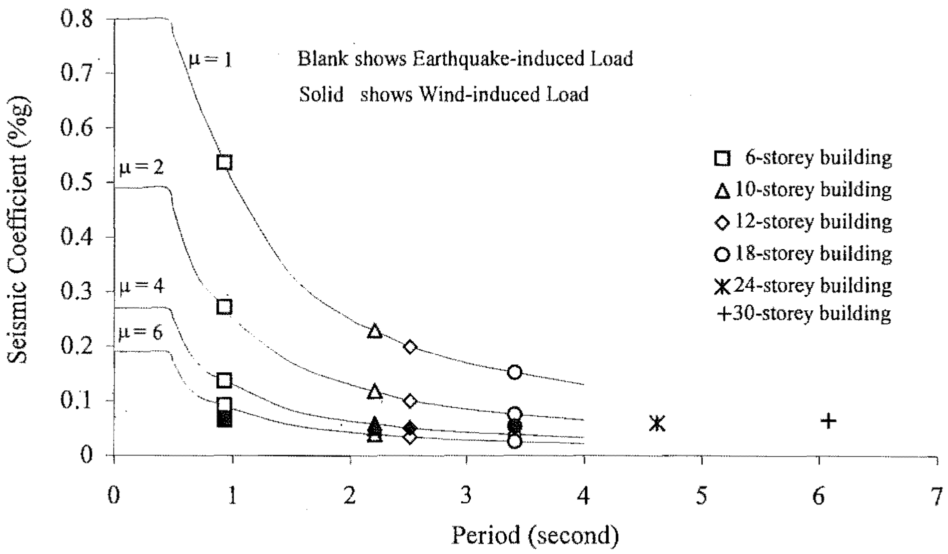


Fig. 6.2 The Relation of Fundamental Period to Seismic Coefficient

respectively. The yield strengths F_y are respectively 3% and 5%W for the structures designed to NZS 3101:1995 and NZS 3101:1982, where W is the total weight of the structure. Furthermore, the same procedures were used for a 6-storey building for which the fundamental period of these structures on a fixed base $T_{1(UI)}$ varies from 0.2 to 1.2 seconds to verify the results obtained from 12-storey multistorey building.

Following the approach suggested by Turkington et al (1987), the effective fundamental period of the building is evaluated directly from the time history plots. This was obtained by measuring the period of the half cycle immediately before and after the peak response and by doubling the period of the half cycle immediately before the peak response.

In order to evaluate the suitability of the effective period for determining seismic response, the measured effective period was compared with the period from the free-vibration modal analyses based on the effective secant stiffness of the base isolation system at the peak response. In this approach, the effective fundamental period is calculated by taking into account the mass and stiffness of the whole structure. It can be seen that the calculated effective period obtained from free-vibration modal analyses is in good agreement with the measured value as shown in Tables 6.1 and 6.2.

6.2.4 Determination of Effective Damping

As suggested by Turkington et al (1987), the additional equivalent viscous damping coefficient C_{eff} and the additional damping ratio λ_{eff} were calculated from the time history analyses as follows:

$$C_{eff} = \frac{W_d}{\pi \, \omega_{eff} \, (X_{max}^2)} \tag{6.3}$$

$$\lambda_{eff} = \frac{C_{eff}}{2 \, \omega_{eff} \, M} \tag{6.4}$$

where $\omega_{eff} = 2\pi/T_{eff}$ and T_{eff} is the effective period obtained above, W_d is the work done for the energy dissipated in the base isolator (area of hysteretic loop) at the peak displacement X_{max} and

$T_{1 (UI)}$ (second)	Effective Fundamental Period (second)			Additional Damping (% critical)	
	Calculated *	Measured **			
		Elasto-Plastic	Bilinear	Elasto-Plastic	Bilinear
0.2	0.32	0.45	0.45	12.3	14.4
0.4	0.42	0.60	0.58	11.4	15.6
0.6	0.62	0.75	0.80	10.2	20.2
0.8	0.83	0.90	0.95	9.8	17.6
1.0	1.04	1.20	1.27	9.6	15.1
1.2	1.25	1.35	1.42	8.8	13.2
1.4	1.46	1.50	1.57	8.5	13.4
1.6	1.66	1.88	1.88	8.6	13.8
1.8	1.87	1.95	2.02	8.2	14.6
2.0	2.06	2.18	2.18	8.4	14.8

Note:

* Based on Free-Vibration Modal Analysis of the Whole Structure.
** Measured from Displacement Response History Based on Elasto-Plastic and Bilinear Models.

Table 6.1 Evaluation of Effective Fundamental Period and Additional Damping for 12-Storey Structures Designed to NZS 3101:1995

$T_{1(UI)}$ (second)	Effective Fundamental Period (second)			Additional Damping (% critical)	
	Calculated *	Measured **			
		Elasto-Plastic	Bilinear	Elasto-Plastic	Bilinear
0.2	0.36	0.45	0.45	23.2	30.1
0.4	0.42	0.60	0.60	23.4	30.2
0.6	0.63	0.83	0.90	22.5	32.1
0.8	0.84	0.98	1.05	22.8	32.6
1.0	1.06	1.20	1.27	16.8	27.4
1.2	1.26	1.35	1.42	18.4	25.5
1.4	1.47	1.58	1.65	16.4	22.6
1.6	1.67	1.80	1.87	16.6	23.2
1.8	1.88	1.95	1.95	14.7	23.5
2.0	2.08	2.18	2.18	14.8	23.8

Note:

* Based on Free-Vibration Modal Analysis of the Whole Structure.
** Measured from Displacement Response History Based on Elasto-Plastic and Bilinear Models.

Table 6.2 Evaluation of Effective Fundamental Period and Additional Damping for 12-Storey Structures Designed to NZS 3101:1982

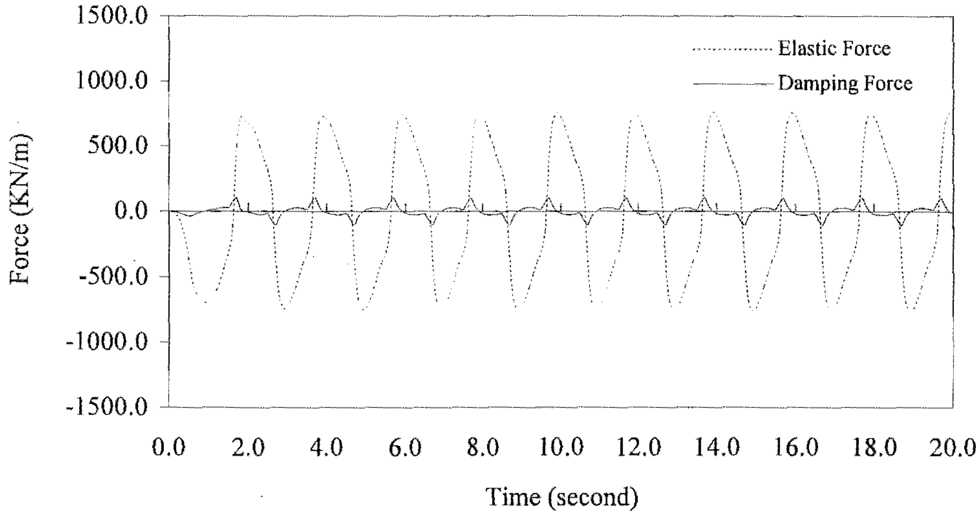
M is mass of the structure. It must be noted that the effective damping is related to the energy dissipated in the base isolators and not to the natural viscous damping in the superstructure.

From Eq. 6.3, the equivalent viscous damping coefficients, C_{eff}, are computed from the cyclic loading time history analyses and are approximately 1208 and 1429 KN/m/sec at the fundamental period, $T_{1\,(UI)}$, of 2.0 second for the elasto-plastic and bilinear models of the isolation systems in the structures designed to NZS 3101:1995. For the structures designed to NZS 3101:1982, the equivalent viscous damping coefficients, C_{eff}, are respectively 1587 and 1796 KN/m/sec for the elasto-plastic and bilinear models of the isolation systems.

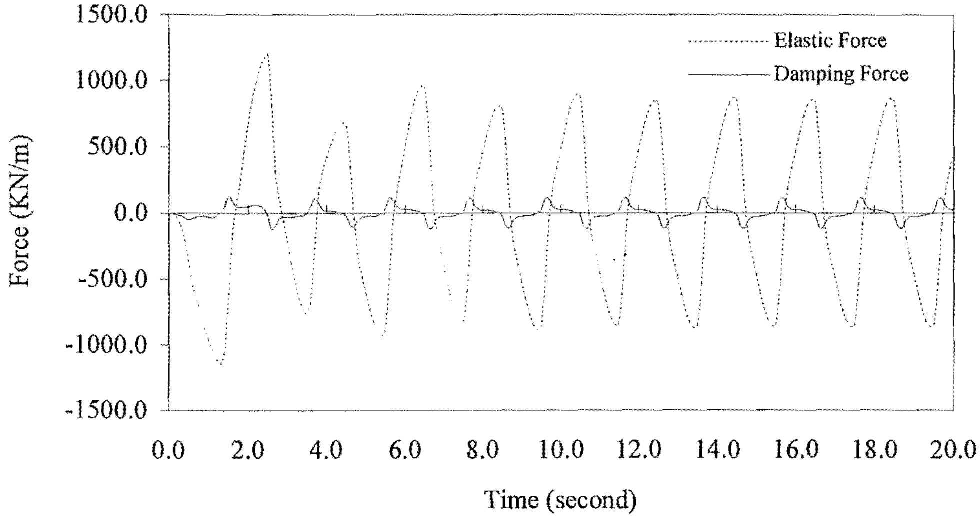
From the installation of base isolation system in the one-dimension uniform model mentioned in Section 3.4.3, the effective stiffness of the base isolator, k_{eff}, can be computed from the energy dissipated in the base isolation system. The effective stiffnesses are approximately 6426 and 6383 KN/m at the fundamental period, $T_{1\,(UI)}$, of 2.0 second for the elasto-plastic and bilinear models in the structures designed to NZS 3101:1995. This is much smaller than the column stiffness, 225060 KN/m of the uniform structure. Also, for the structures designed to NZS 3101:1982, the effective stiffnesses of the base isolators are respectively 9215 and 9058 KN/m for the elasto-plastic and bilinear models and are much smaller than the column stiffness, 250560 KN/m of the uniform structure.

Fig. 6.3 shows that the damping forces (127 KN) are much smaller than the elastic forces (1193 KN) in the uniform models with additional damping devices for the cyclic loading time history analyses of the structures designed to NZS 3101:1995. Similar results are seen in the time history analyses under the El Centro 1940 N-S earthquake as shown in Fig. 6.4. This is true for the structures designed to NZS 3101:1982.

The additional damping ratios were computed from the time history analyses using Eq. 6.4. They are added to the nominal 5% included in the analyses to account for inherent damping in the structure as indicated in Ref. C3. The effective damping ratio is the sum of the hysteretic damping ratio and 5%. The expected additional damping due to the hysteretic behaviour of the base isolation system based on elasto-plastic and bilinear models from two different design standards are listed in Tables 6.1 and 6.2 respectively.

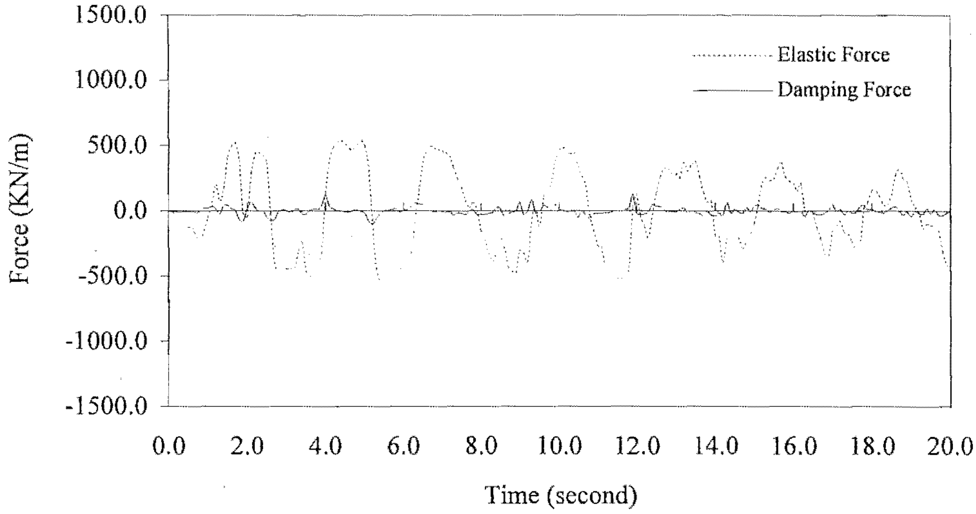


(a) Structures Mounted on Elasto-Plastic Isolation Systems

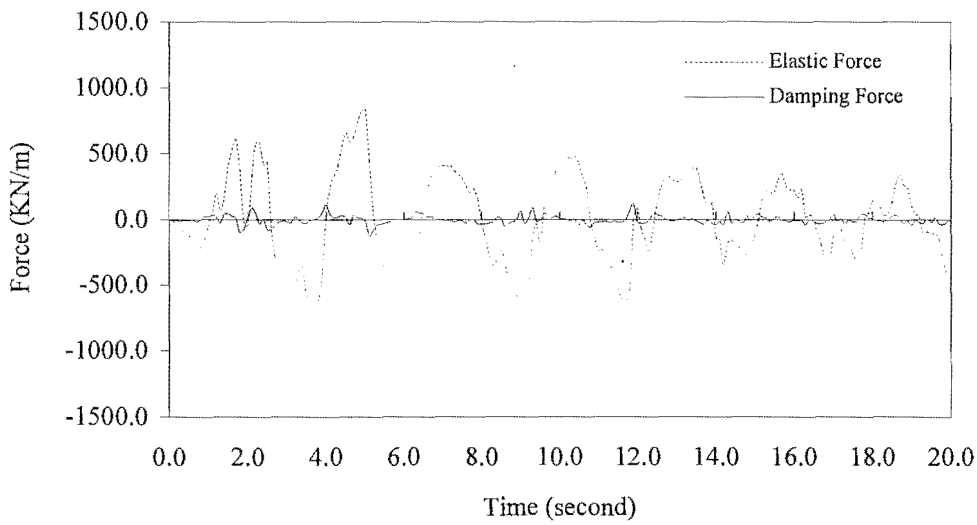


(b) Structures Mounted on Bilinear Isolation Systems

Fig. 6.3 Comparisons of Elastic and Damping Forces for Structures Designed to NZS 3101:1995 under Cyclic Loading Analyses



(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems

Fig. 6.4 Comparisons of Elastic and Damping Forces for Structures Designed to NZS 3101:1995 under the El Centro 1940 N-S Earthquake

Further, these values of calculated total equivalent viscous damping of all structures, with $T_{1\,(UI)}$ from 0.2 to 2.0 seconds discussed above, were then given their limits using the critical damping measured from the displacement spectrum of the El Centro 1940 N-S earthquake as shown in Figs. 6.5 and 6.6, respectively.

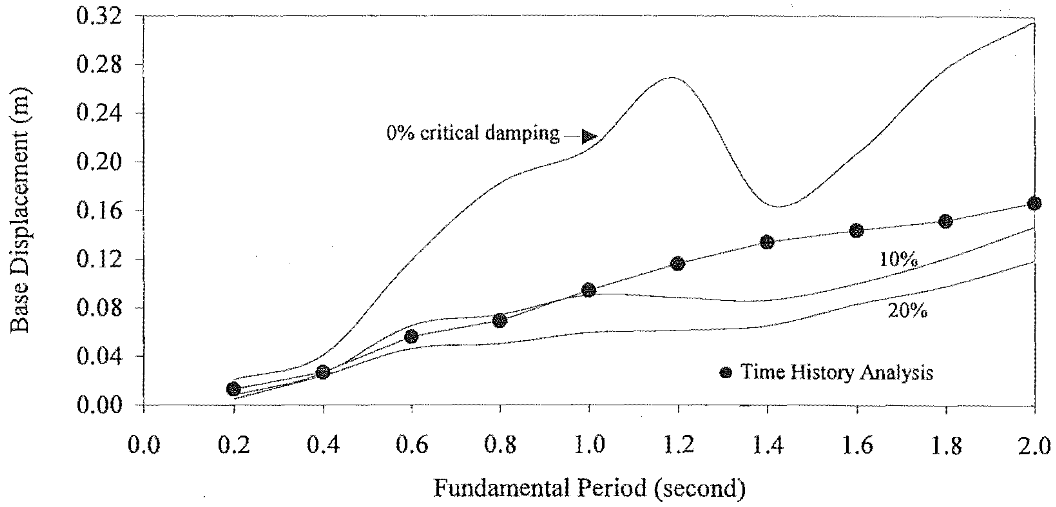
In a modal response spectrum analysis, the base shear V_i in the ith mode of a structure can be obtained from the spectral acceleration S_A (Carr 1994) using the following equation:

$$V_{i\ max} = (\text{ Effective Weight })_i \left(\frac{S_A}{g}\right)_i \tag{6.5}$$

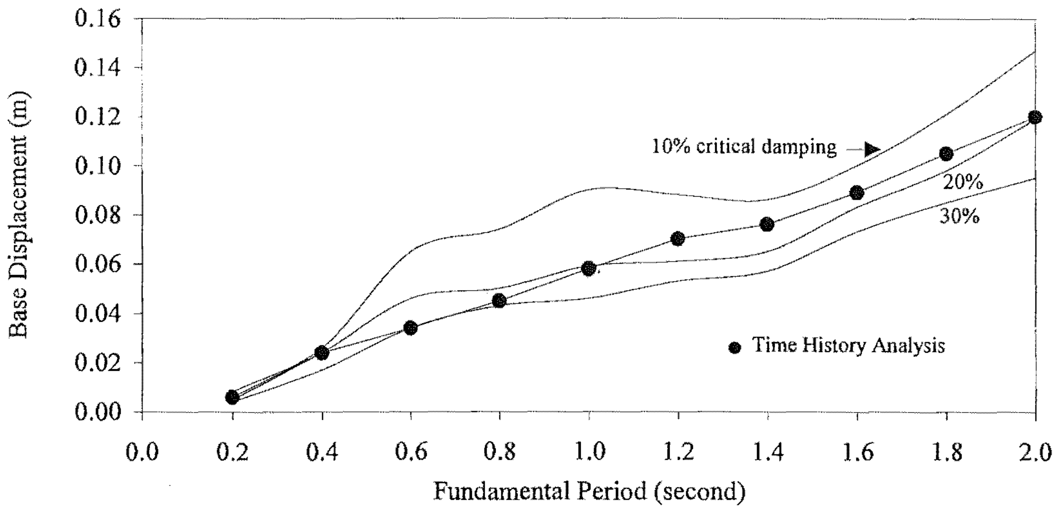
where the effective weight of the ith mode indicates the importance of the contribution of the ith mode to the total base shear acting on the structure. Based on Section 4.9.1.2 of NZS 4203:1992, the effective weight must be at least 90% of the total weight of the structure when modal response spectrum analyses are used. Using Eq. 6.5, the base shears of the structures for the critical damping ratios can then be measured from the acceleration spectrum of the earthquake.

The base shears and shear forces of the base isolation system for the base isolated structures obtained from loading time history analyses, with $T_{1\,(UI)}$ from 0.2 to 2.0 seconds, were compared with the base shears for unisolated structures determined using the acceleration spectrum of the El Centro 1940 N-S earthquake as shown in Figs. 6.7 and 6.8 respectively. As can be seen from Figs. 6.7 and 6.8, the base shears of the base isolated structures decrease due to the installation of inelastic base isolation systems, though the superstructure remains elastic. This occurs for both structures with elasto-plastic and with bilinear isolation systems and designed to NZS 3101:1995 or NZS 3101:1982.

In the case of a 6-storey building on a fixed base having a fundamental period $T_{1\,(UI)}$ ranging from 0.2 to 1.2 seconds, the effective fundamental period of the base isolated structures with elasto-plastic and bilinear isolators are shown in Table 6.3. The values of total equivalent damping of all these buildings, with $T_{1\,(UI)}$ from 0.2 to 1.2 seconds, using the critical damping measured from the displacement and acceleration spectra of the El Centro 1940 N-S earthquake are shown in Figs. 6.9 and 6.10 respectively. From the results obtained above, it can be seen that the expression of effective fundamental period and total equivalent viscous damping of a 6-storey base isolated structure is similar to that presented for the 12-storey base isolated building.

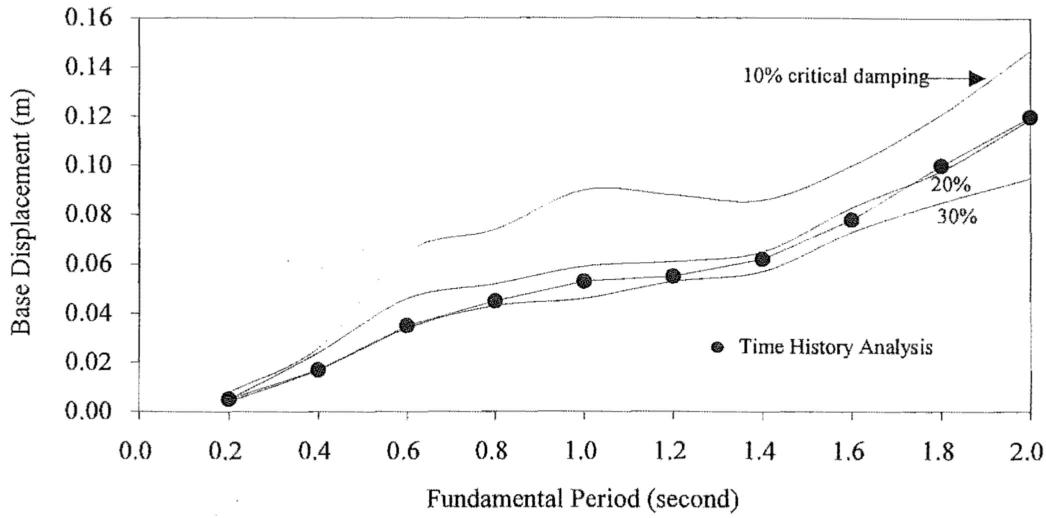


(a) Elasto-Plastic Model of the Isolation Systems

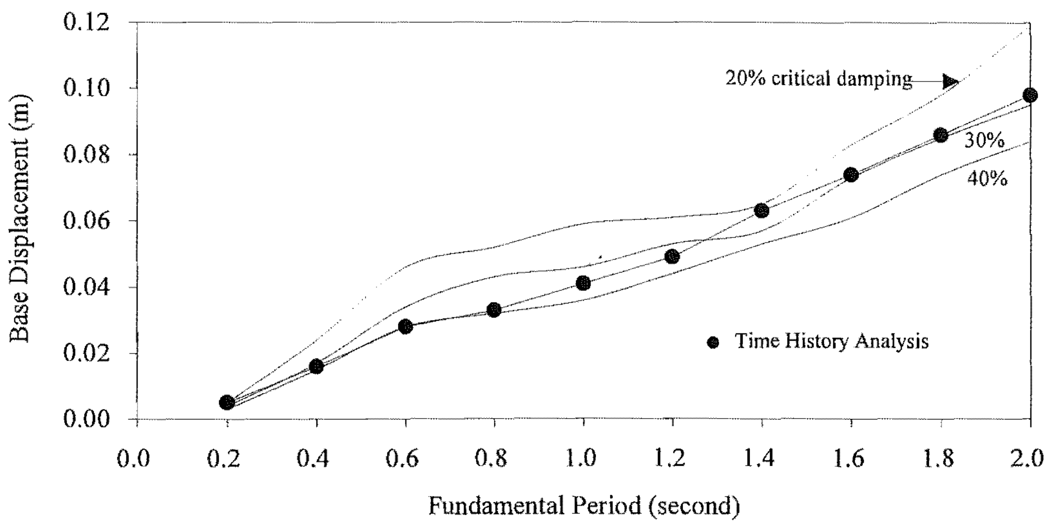


(b) Bilinear Model of the Isolation Systems

Fig. 6.5 Limits of Base Displacement shown by the Approximate Method for 12-Storey Structures Designed to NZS 3101:1995

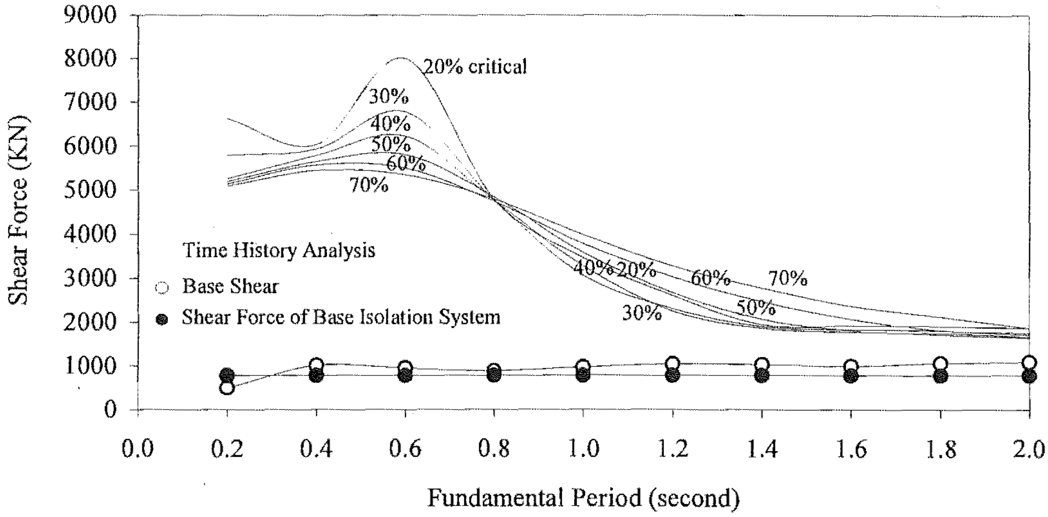


(a) Elasto-Plastic Model of the Isolation Systems

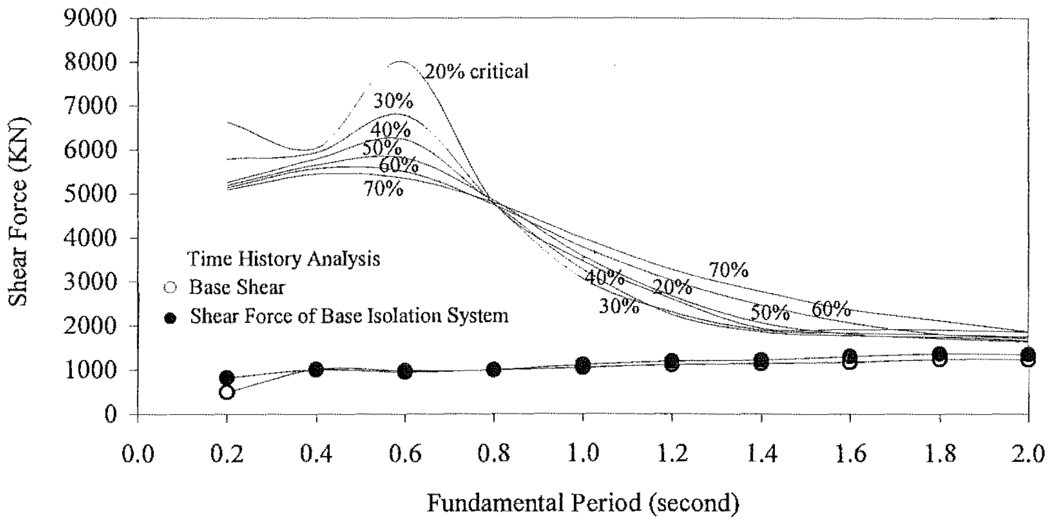


(b) Bilinear Model of the Isolation Systems

Fig. 6.6 Limits of Base Displacement shown by the Approximate Method for 12-Storey Structures Designed to NZS 3101:1982

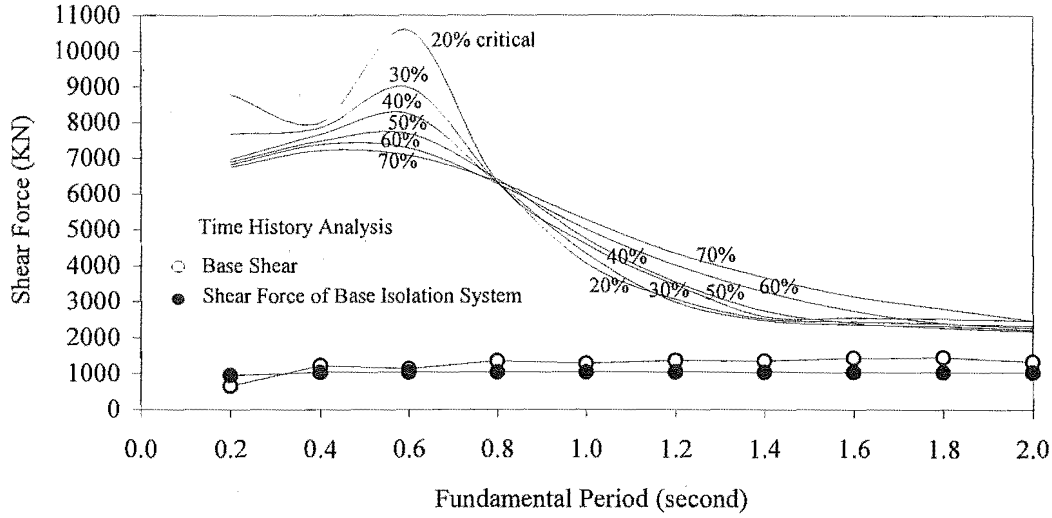


(a) Elasto-Plastic Model of the Isolation Systems

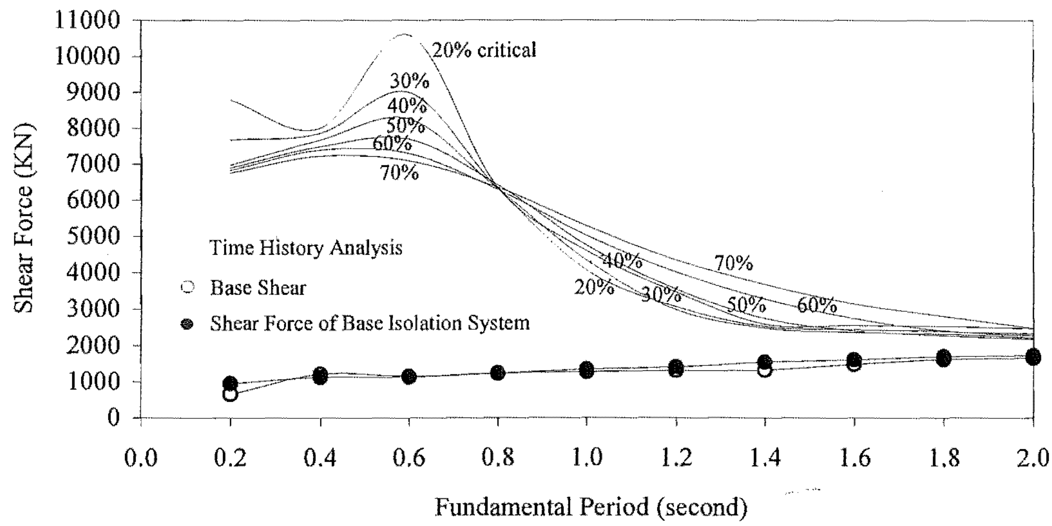


(b) Bilinear Model of the Isolation Systems

Fig. 6.7 Limits of Shear Forces shown by the Approximate Method for 12-Storey Structures Designed to NZS 3101:1995



(a) Elasto-Plastic Model of the Isolation Systems



(b) Bilinear Model of the Isolation Systems

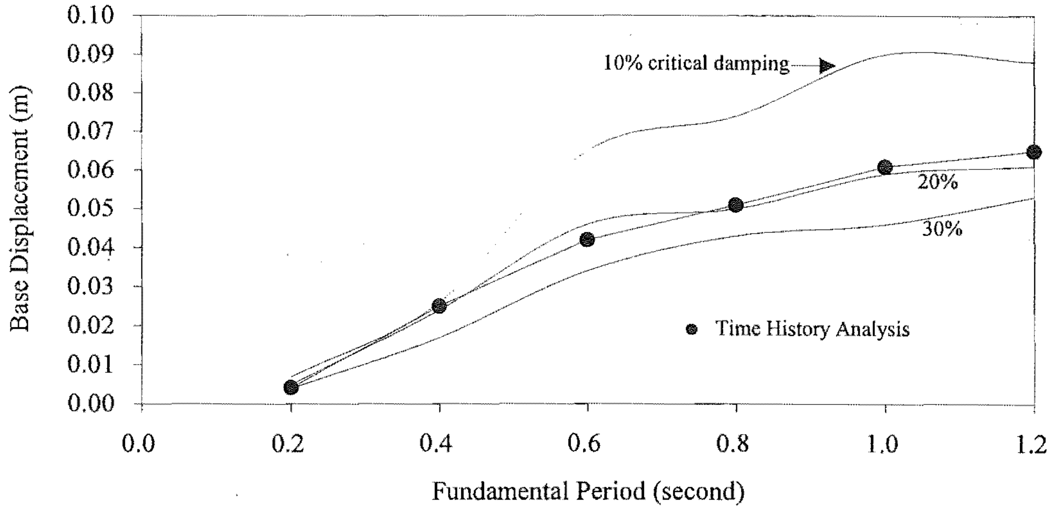
Fig. 6.8 Limits of Shear Forces shown by the Approximate Method for 12-Storey Structures Designed to NZS 3101:1982

$T_{1\,(UI)}$ (second)	Effective Fundamental Period (second)			Additional Damping (% critical)	
	Calculated*	Measured **			
		Elasto-Plastic	Bilinear	Elasto-Plastic	Bilinear
0.2	0.36	0.60	0.60	14.6	21.2
0.4	0.43	0.60	0.60	15.2	21.8
0.6	0.65	0.84	0.75	16.2	23.9
0.8	0.86	0.90	0.90	15.1	24.1
1.0	1.08	1.20	1.20	14.5	23.2
1.2	1.29	1.50	1.35	14.7	23.6

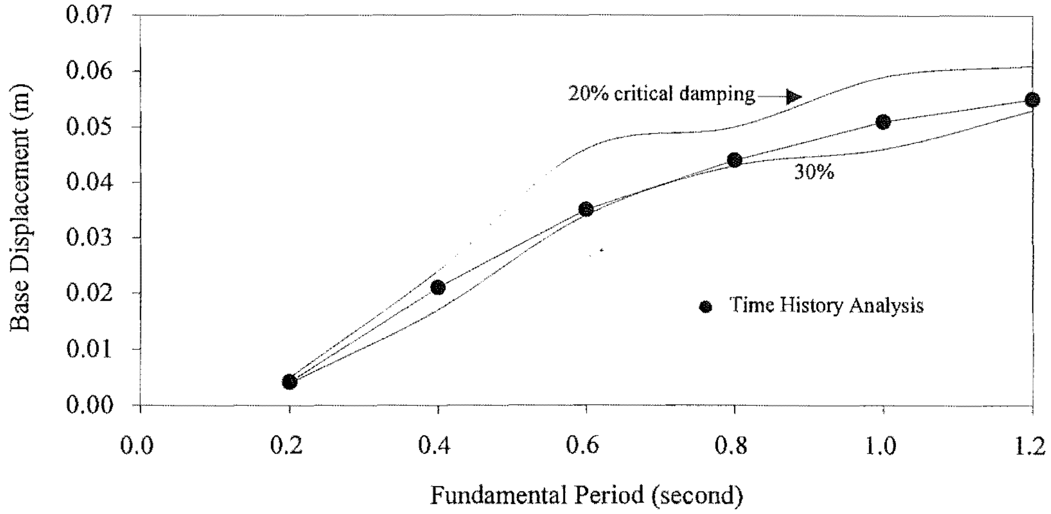
Note:

* Based on Free-Vibration Modal Analysis of the Whole Structure.
** Measured from Displacement Response History Based on Elasto-Plastic and Bilinear Models.

Table 6.3 Evaluation of Effective Fundamental Period and Additional Damping for 6-Storey Structures

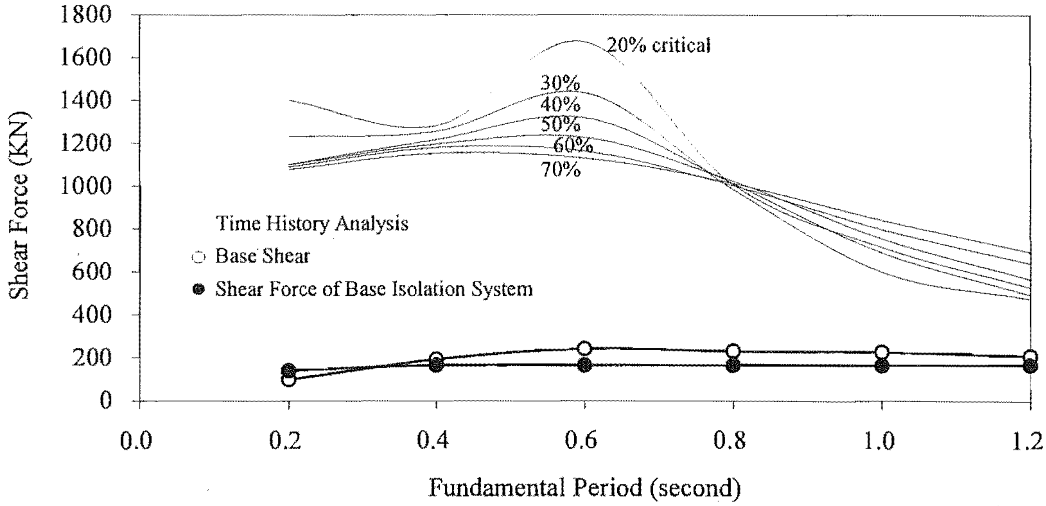


(a) Elasto-Plastic Model of the Isolation Systems

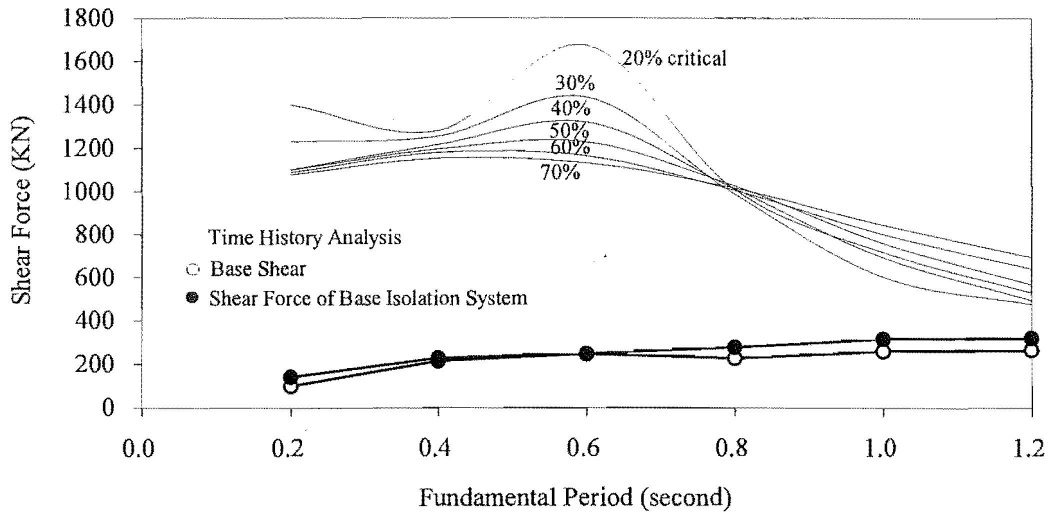


(b) Bilinear Model of the Isolation Systems

Fig. 6.9 Limits of Base Displacement shown by the Approximate Method for 6-Storey Structures



(a) Elasto-Plastic Model of the Isolation Systems



(b) Bilinear Model of the Isolation Systems

Fig. 6.10 Limits of Shear Forces shown by the Approximate Method for 6-Storey Structures

6.3 Seismic Performances of Structures with Additional Damping

6.3.1 General

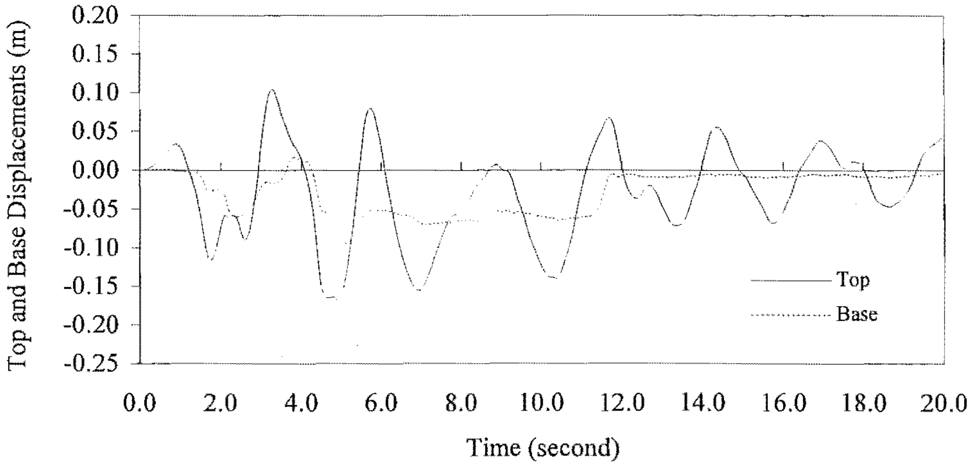
In order to assure the suitability of velocity-dependent damping systems for improving the seismic resistance of buildings, the seismic behaviour of the different 12-storey buildings with the equivalent viscous damping coefficient as obtained from Section 6.2.4 using the uniform superstructure models mentioned in Section 3.4.3, subjected to the N-S component of El Centro 1940 earthquake will be investigated.

In this section, the structures were designed to NZS 3101:1995. The structural response parameters, such as lateral storey displacements, interstorey drifts, total acceleration, base shears and lateral storey shear envelopes obtained from the time history analyses, are presented in order to show the typical performance of the base isolated structures.

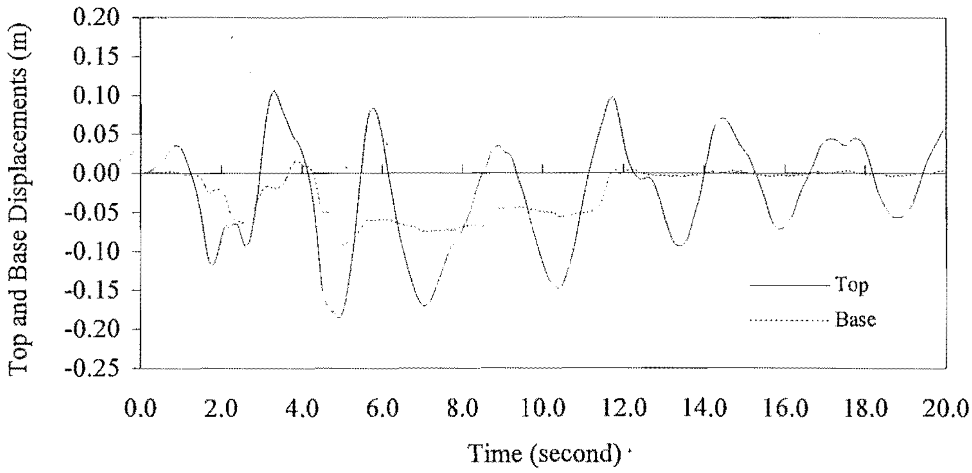
6.3.2 Lateral Storey Displacements and Interstorey Drifts

As shown in Fig. 6.11, the top floor displacement for the fixed base building with additional damping is 20% less than that without additional damping as given in Fig. 5.3. For the structures mounted on elasto-plastic isolation systems, the top floor displacements relative to their base floor displacements of the base isolated and segmental structures with additional damping on a rigid base and a compliant foundation show approximately 10%, 14%, 19% and 18% less than those without additional damping, respectively.

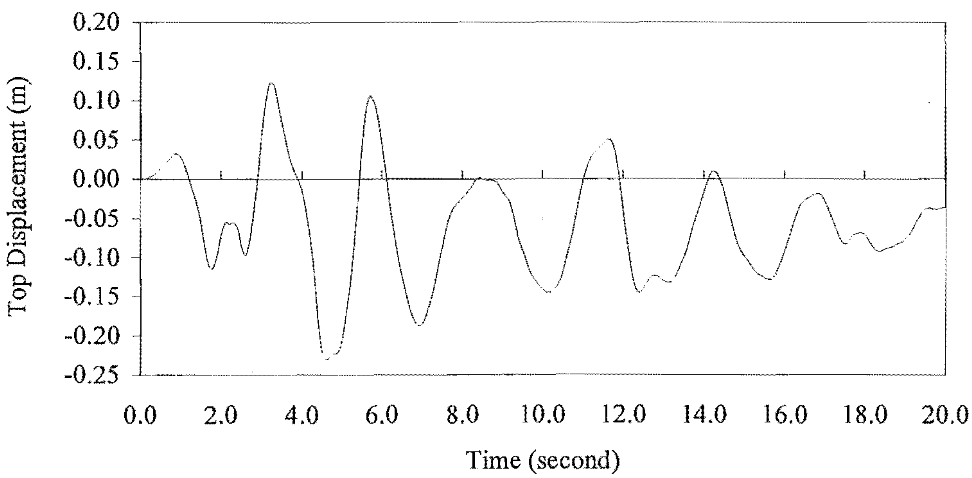
From the responses of the structures with bilinear isolation systems shown in Fig. 6.12, the base isolated and segmental structures with additional damping on a rigid base and a compliant foundation show that the top floor displacements relative to their base floor displacements are approximately 8%, 9%, 12% and 8% less than those without additional damping as shown in Fig. 5.4. As mentioned above, the structures with additional damping ratio (8% critical) using elasto-plastic isolation devices have a greater reduction of displacement than the same buildings with additional damping ratio (15% critical) using bilinear isolation systems. This reduction of response would be expected as the damping in the structure increases. These



(a) Base Isolated Building with Rigid Base

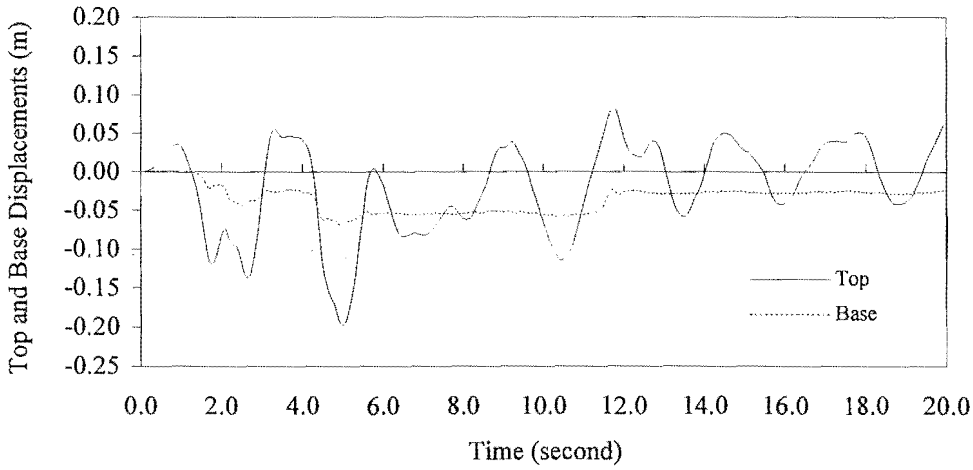


(b) Base Isolated Building with Foundation Compliance

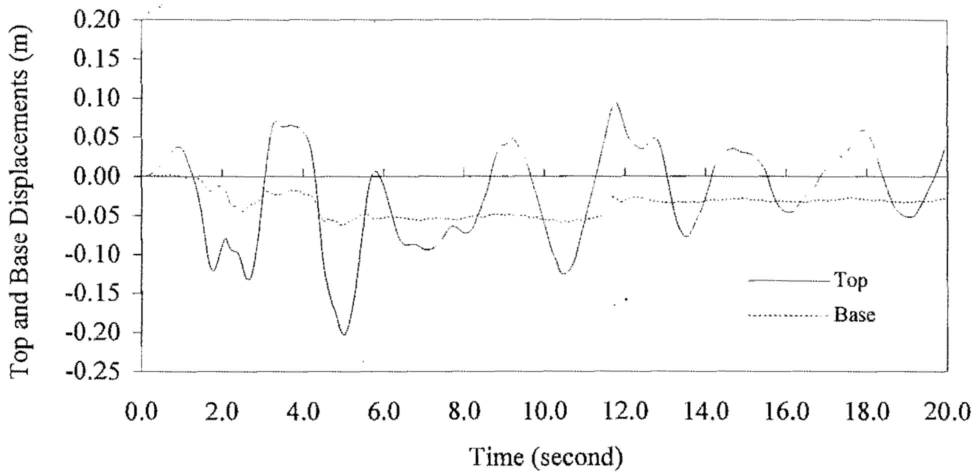


(c) Fixed Base Building

Fig. 6.11 The Response History of Top and Base Floor Displacements for Structures with Additional Damping Using Elasto-Plastic Isolation Systems

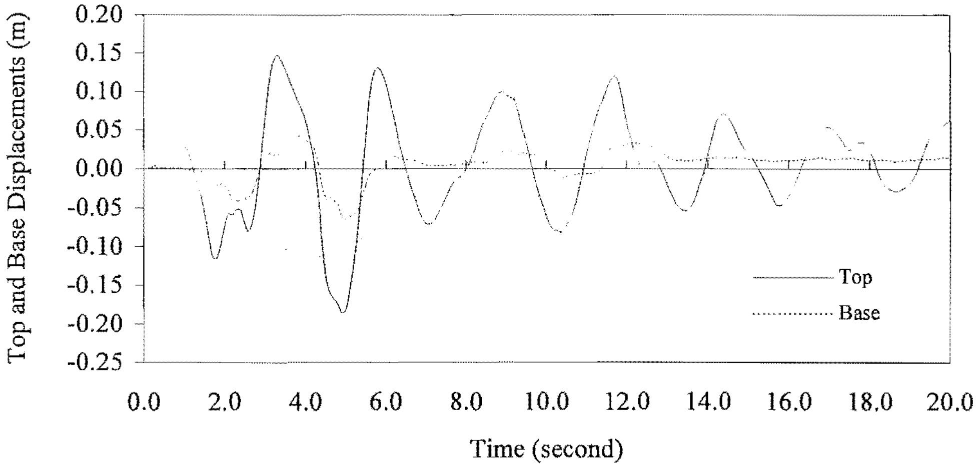


(d) Segmental Building with Rigid Base

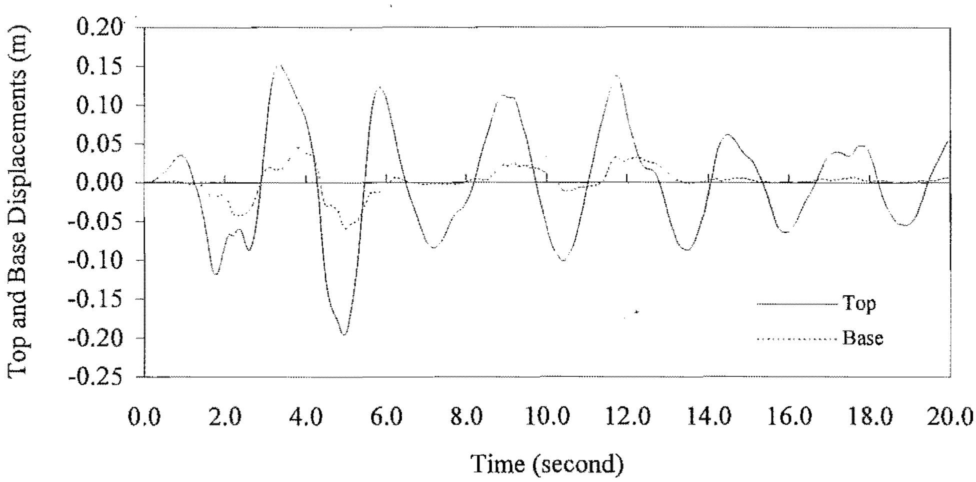


(e) Segmental Building with Foundation Compliance

Fig. 6.11 ..... (continued)

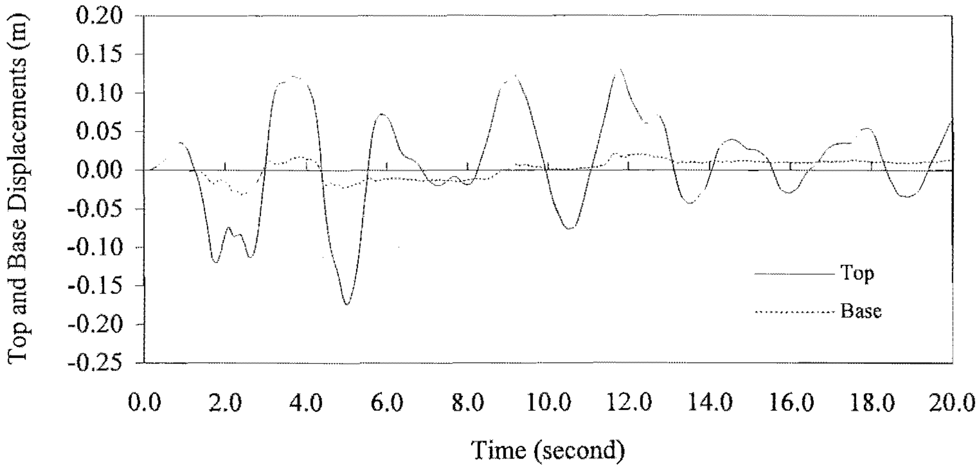


(a) Base Isolated Building with Rigid Base

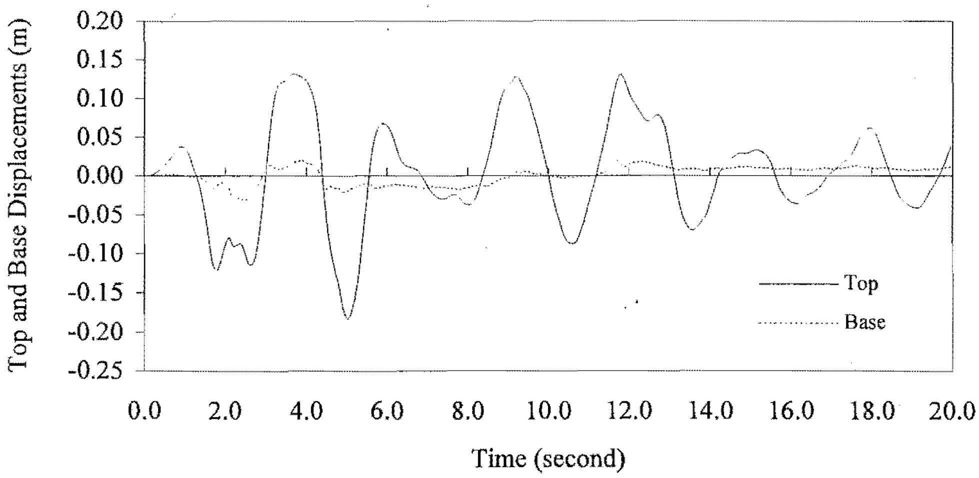


(b) Base Isolated Building with Foundation Compliance

Fig. 6.12 The Response History of Top and Base Floor Displacements for Structures with Additional Damping Using Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation Compliance

Fig. 6.12 ..... (continued)

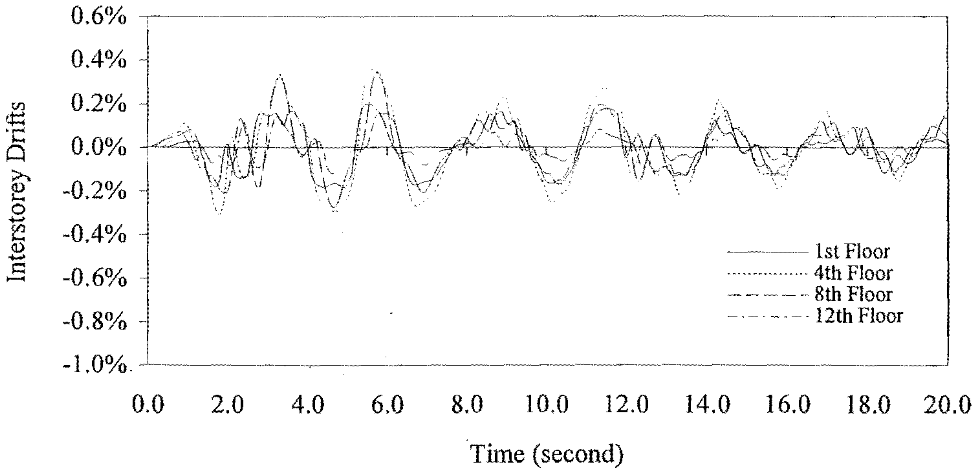
results show that the increased additional damping due to the hysteretic behaviour of the isolation devices reduces displacements for the base isolated and segmental structures.

The results of the structures with additional damping mounted on elasto-plastic isolation devices are shown in Fig. 6.11, the fixed base structure has a greater top floor displacement than that for the base isolated and segmental building during the twenty seconds of excitation. This is similar to the responses obtained for the structures with additional damping using bilinear isolation systems as shown in Fig. 6.12.

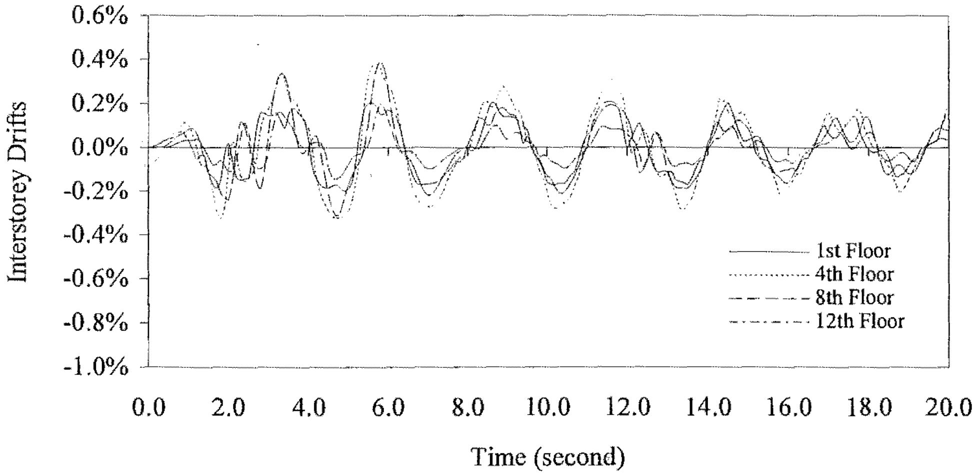
Compared with the base floor displacement responses shown in Fig. 6.11, the base isolated building with a rigid base shows a greater response in the first ten seconds, though the dynamic characteristics do not change dramatically in that the overall natural period appears reasonably constant. After ten seconds, the segmental building with a rigid base shows the greatest response, though this is less than that which occurred for the base isolated building with a rigid base. Comparing the base floor displacement responses of Fig. 6.12, the base isolated building with a rigid base shows the greatest displacement response during the twenty seconds of excitation.

From the responses of the buildings with additional damping mounted on elasto-plastic isolators shown in Fig. 6.13, the segmental buildings have smaller interstorey drifts than the base isolated buildings. When compared with the fixed base building, the interstorey drift responses are also smaller. This is similar to responses obtained for the structures with additional damping using bilinear isolators as shown in Fig. 6.14. These results imply that the segmental buildings make very significant contributions in the reduction of interstorey drifts. The much smaller interstorey drifts prevent the occurrence of non-structural damage during moderate earthquake.

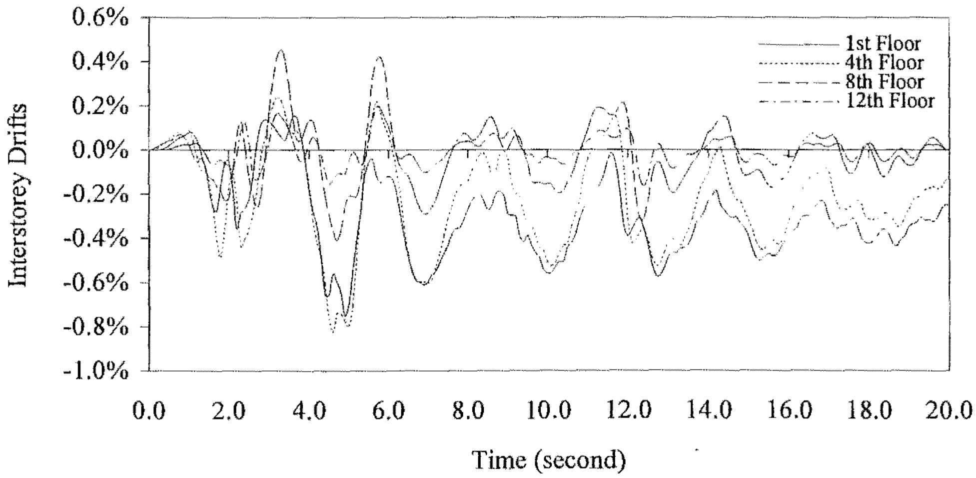
The interstorey drifts for the different structures are summarised in Table 6.4. The fixed base building with additional damping has smaller interstorey drifts of less than 1.0%, compared with the fixed base building without additional damping from Table 5.4. This is true for the base isolated and segmental structures with additional damping when the structures are mounted on elasto-plastic and bilinear isolation systems. Similar results are obtained for the structures designed to NZS 3101:1982 as shown in Table B.1 of Appendix B.



(a) Base Isolated Building with Rigid Base

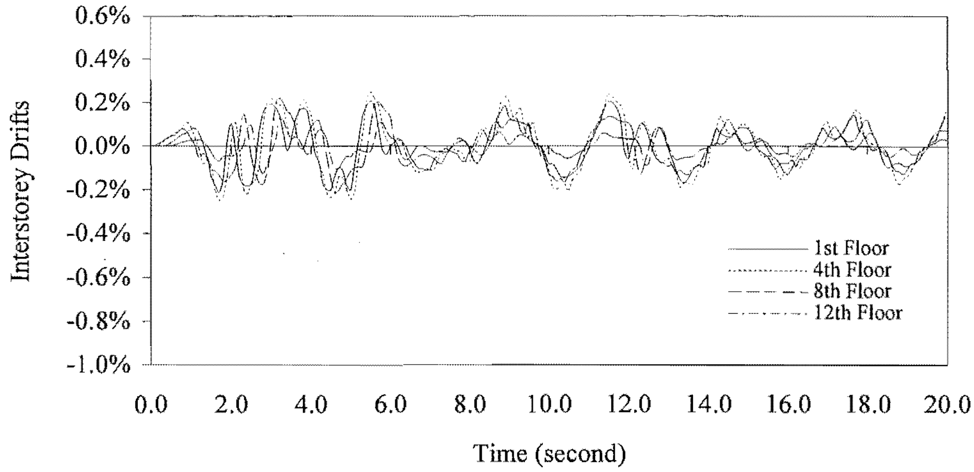


(b) Base Isolated Building with Foundation Compliance

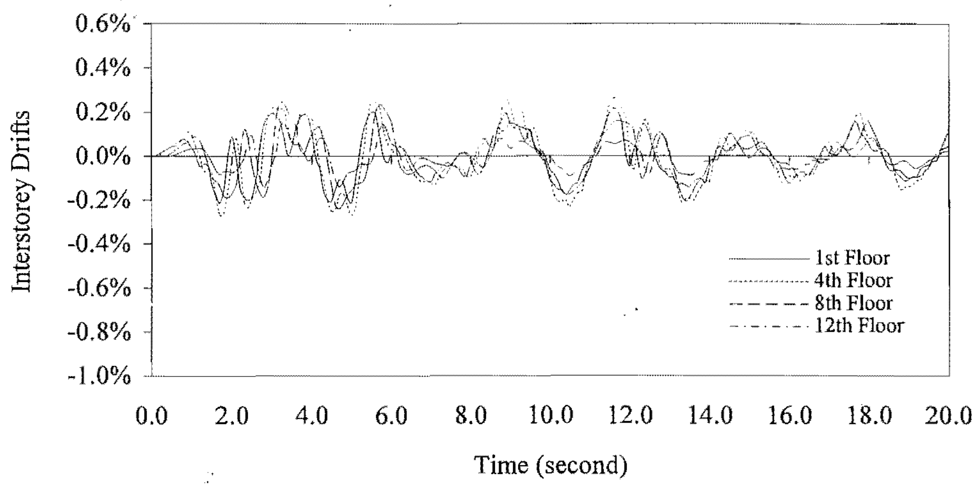


(c) Fixed Base Building

Fig. 6.13 The Response History of Interstorey Drifts for Structures with Additional Damping Using Elasto-Plastic Isolation Systems

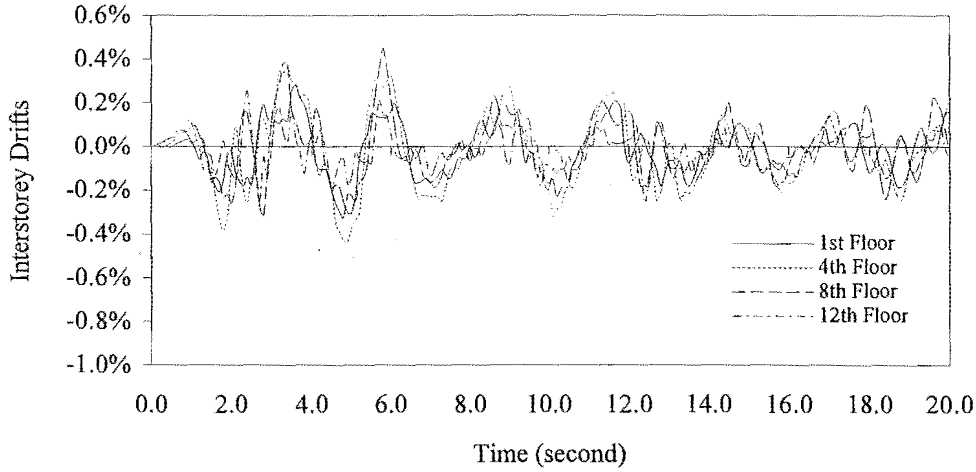


(d) Segmental Building with Rigid Base

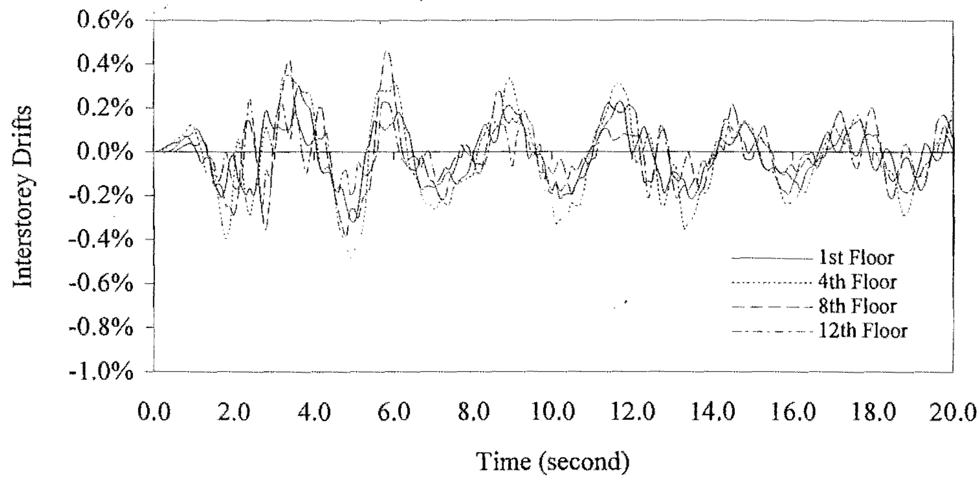


(e) Segmental Building with Foundation Compliance

Fig. 6.13 ..... (continued)

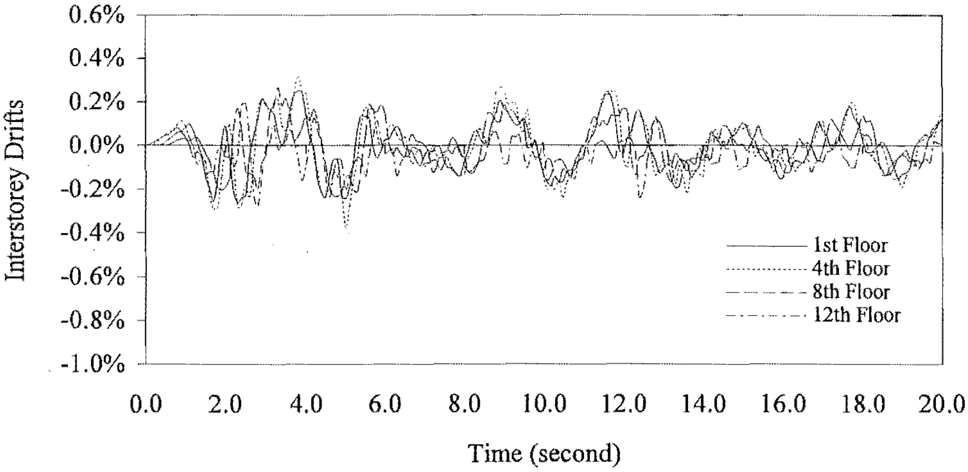


(a) Base Isolated Building with Rigid Base

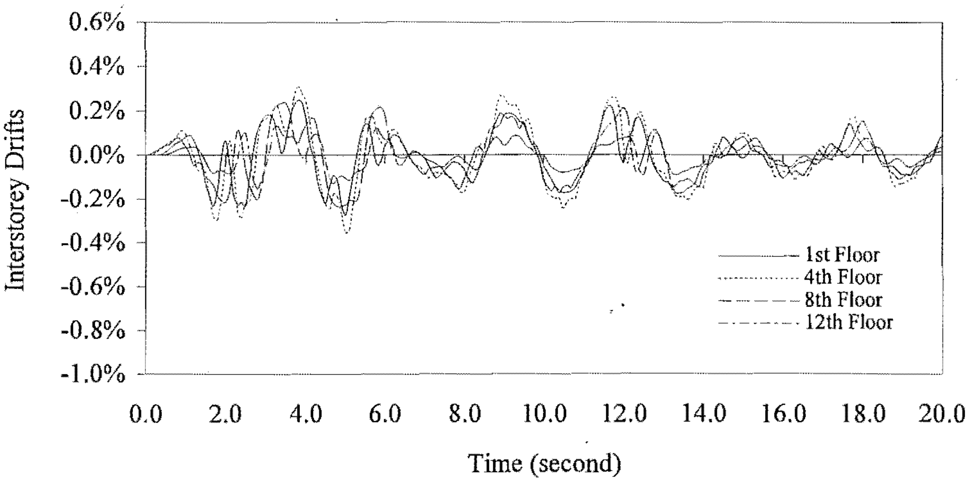


(b) Base Isolated Building with Foundation Compliance

Fig. 6.14 The Response History of Interstorey Drifts for Structures with Additional Damping Using Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation Compliance

Fig. 6.14 ..... (continued)

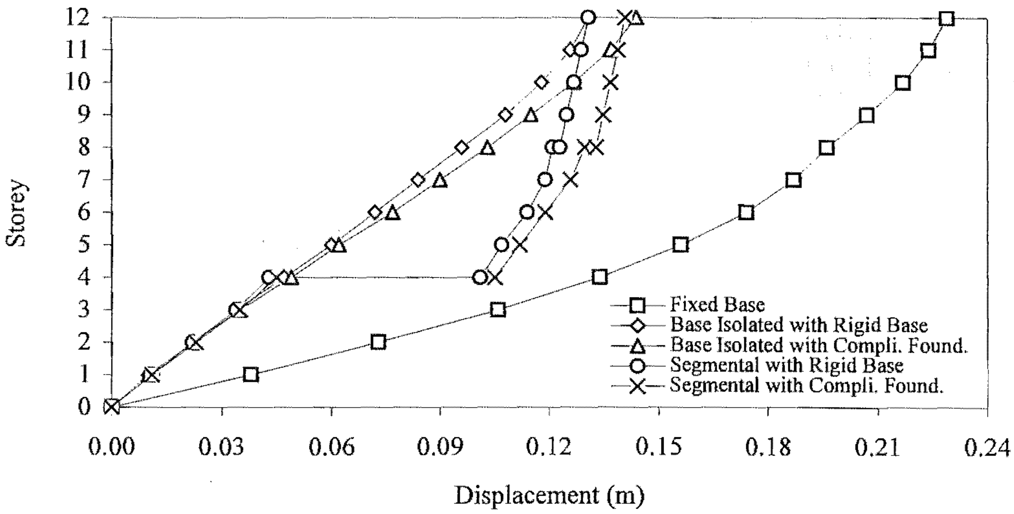
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.14%	0.19%	0.08%	0.08%
11	0.19%	0.22%	0.30%	0.08%	0.08%
10	0.27%	0.27%	0.33%	0.08%	0.08%
9	0.30%	0.33%	0.33%	0.08%	0.08%
8	0.33%	0.33%	0.36%	0.08%	0.11%
7	0.36%	0.33%	0.36%	0.14%	0.19%
6	0.49%	0.33%	0.41%	0.19%	0.19%
5	0.60%	0.36%	0.36%	0.16%	0.19%
4	0.77%	0.36%	0.38%	0.25%	0.27%
3	0.90%	0.33%	0.36%	0.33%	0.33%
2	0.96%	0.33%	0.33%	0.30%	0.33%
1	0.76%	0.20%	0.20%	0.22%	0.22%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

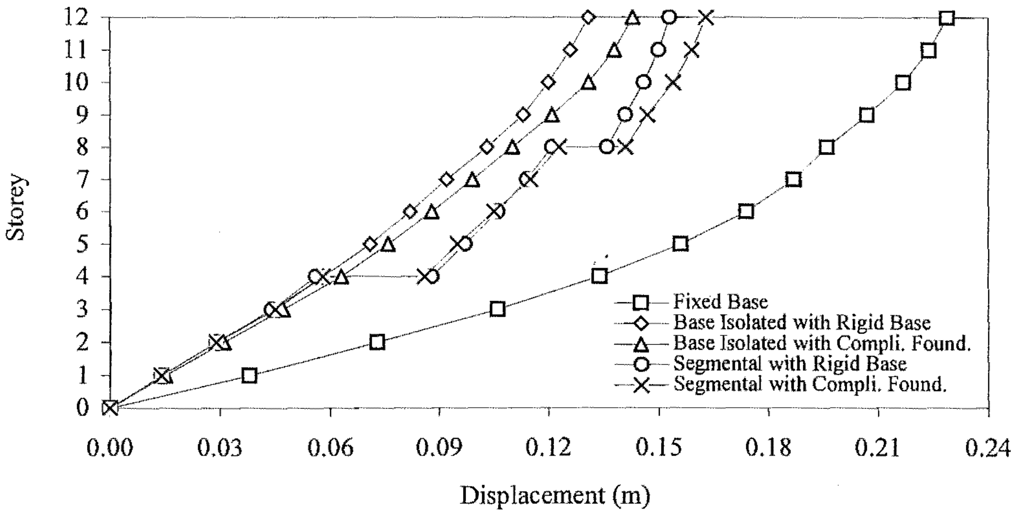
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.14%	0.14%	0.08%	0.11%
11	0.19%	0.16%	0.19%	0.11%	0.14%
10	0.27%	0.19%	0.27%	0.14%	0.19%
9	0.30%	0.27%	0.30%	0.14%	0.16%
8	0.33%	0.30%	0.30%	0.19%	0.22%
7	0.36%	0.27%	0.30%	0.22%	0.27%
6	0.49%	0.30%	0.33%	0.25%	0.27%
5	0.60%	0.36%	0.36%	0.30%	0.25%
4	0.77%	0.38%	0.44%	0.33%	0.36%
3	0.90%	0.41%	0.44%	0.41%	0.44%
2	0.96%	0.41%	0.44%	0.41%	0.41%
1	0.76%	0.28%	0.30%	0.28%	0.28%

(b) Structures Mounted on Bilinear Isolation Systems

Table 6.4 Interstorey Drifts for Different Types of Structures



(a) Elasto-Plastic Model of the Isolation Systems



(b) Bilinear Model of the Isolation Systems

Fig. 6.15 Comparison of Displacement with Storey for Fixed Base, Base Isolated and Segmental Buildings with Additional Damping

When compared with the fixed base structure, the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolation systems have dramatically reduced interstorey displacements as shown in Fig. 6.15. Similar results are also obtained for the buildings designed to NZS 3101:1982 as shown in Fig. B.1 of Appendix B.

6.3.3 Total Acceleration

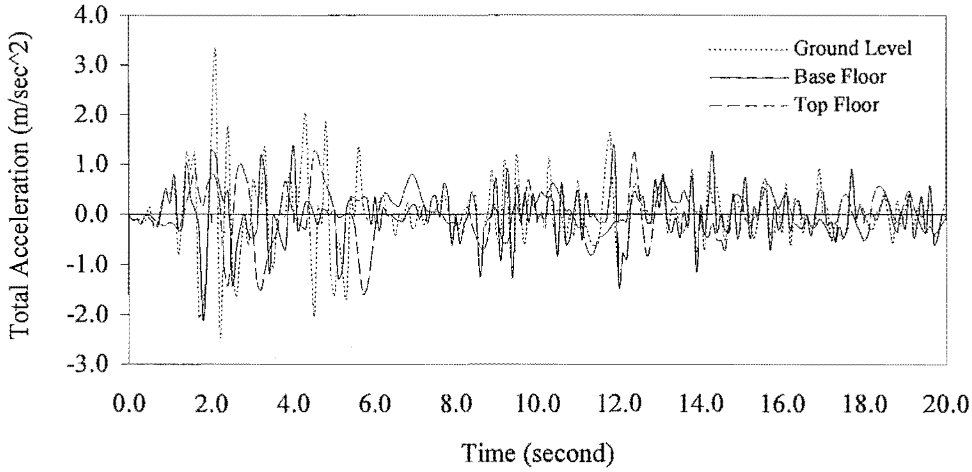
As shown in Fig. 6.16, the total acceleration responses for the base isolated and segmental buildings with additional damping mounted on elasto-plastic isolation systems are smaller than those without additional damping as shown in Fig. 5.8. Similar results are also obtained for the structures with additional damping using bilinear isolation devices as shown in Fig. 6.17 when compared with the buildings without additional damping as given in Fig. 5.9. This agrees that the structures with extra damping will reduce the magnitude of the acceleration response as indicated in Ref. C4.

For the structures with additional damping using elasto-plastic isolators shown in Fig. 6.16, the base isolated and segmental buildings at the base floor have much smaller total acceleration responses compared with those associated with the ground motions. This is also seen in the structures with additional damping mounted on bilinear isolators as shown in Fig. 6.17.

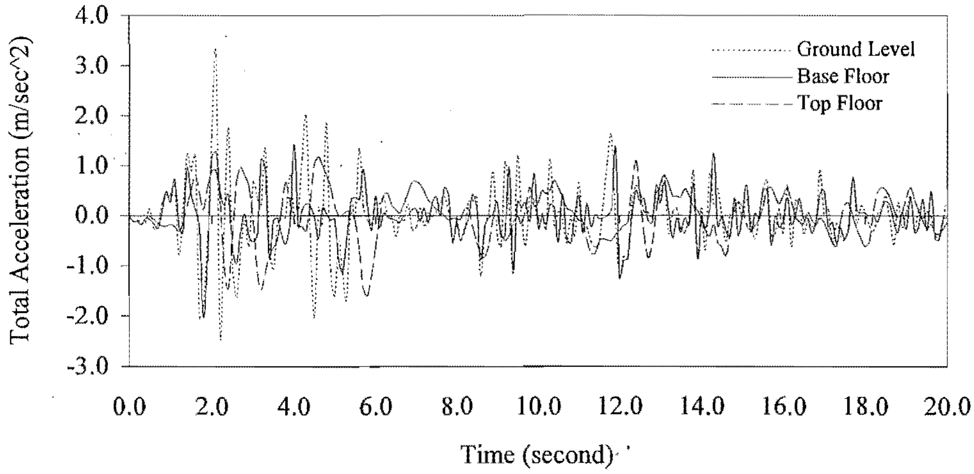
For the structures with additional damping mounted on elasto-plastic isolation systems shown in Fig. 6.16, the base isolated buildings at the base floor show the smallest total acceleration responses, though the segmental buildings have smaller total acceleration responses when compared with the fixed base buildings. Compared with the total acceleration responses, the segmental buildings show smaller total accelerations at the top floor than those observed in the base isolated buildings, and they give smaller total accelerations than the fixed base building. This is similar to results obtained for the structures with additional damping mounted on bilinear isolation systems as shown in Fig. 6.17.

6.3.4 Base Shears and Lateral Storey Shear Envelopes

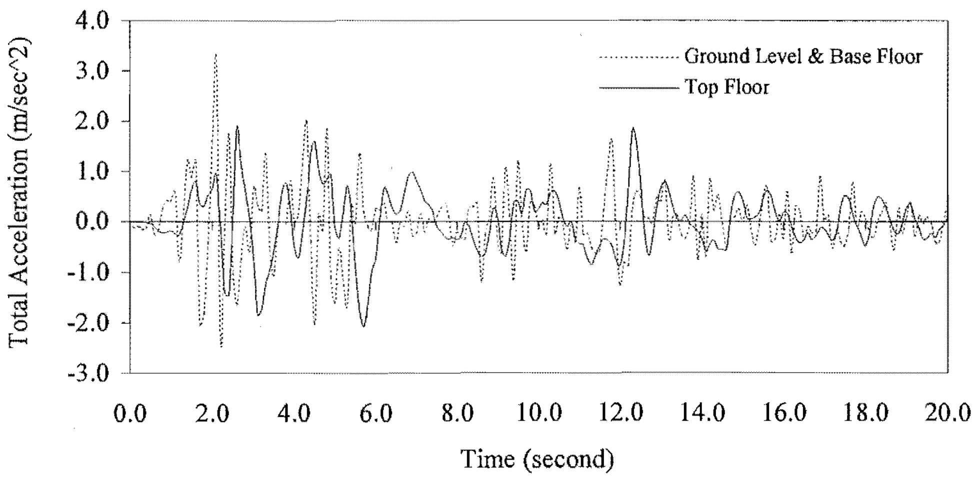
As shown in Fig. 6.18, the base shears of the fixed base, and base isolated and segmental buildings with additional damping on a rigid base and a compliant foundation mounted on



(a) Base Isolated Building with Rigid Base

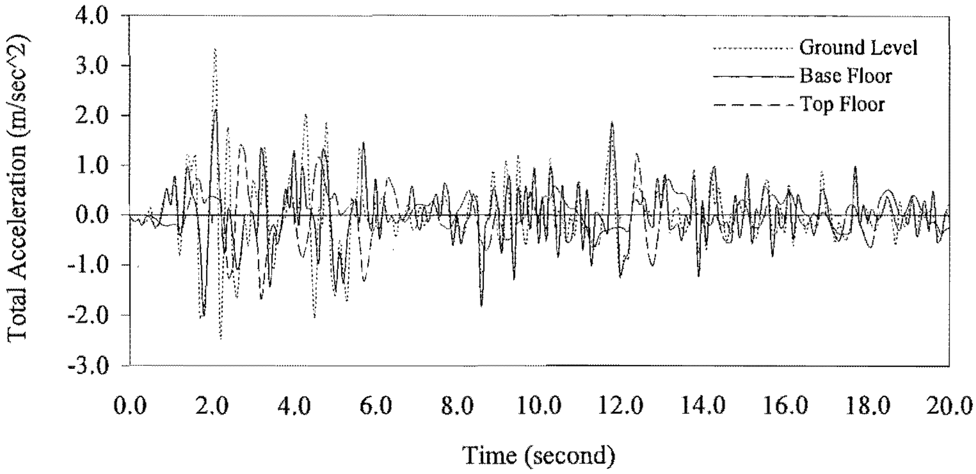


(b) Base Isolated Building with Foundation Compliance

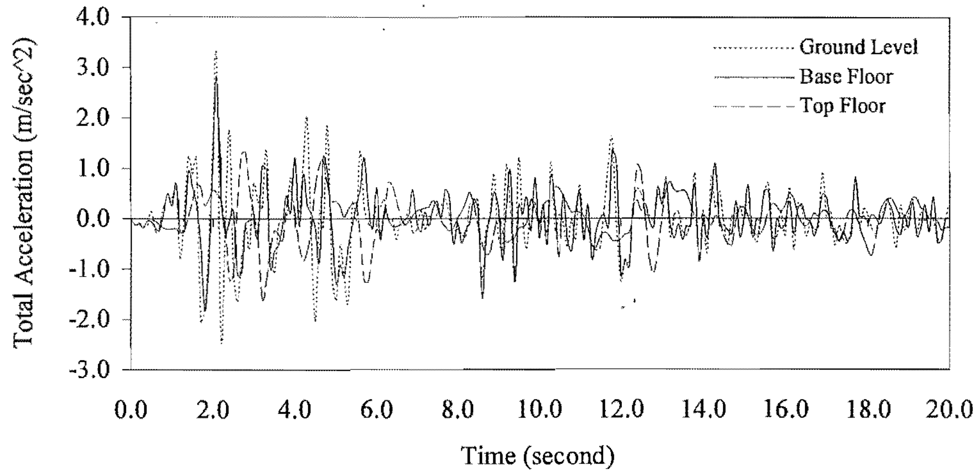


(c) Fixed Base Building

Fig. 6.16 The Response History of Total Accelerations for Structures with Additional Damping Using Elasto-Plastic Isolation Systems

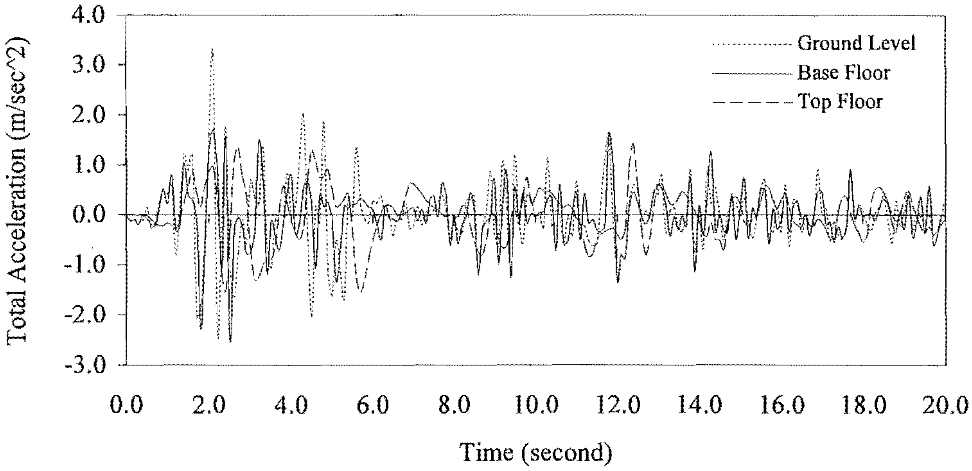


(d) Segmental Building with Rigid Base

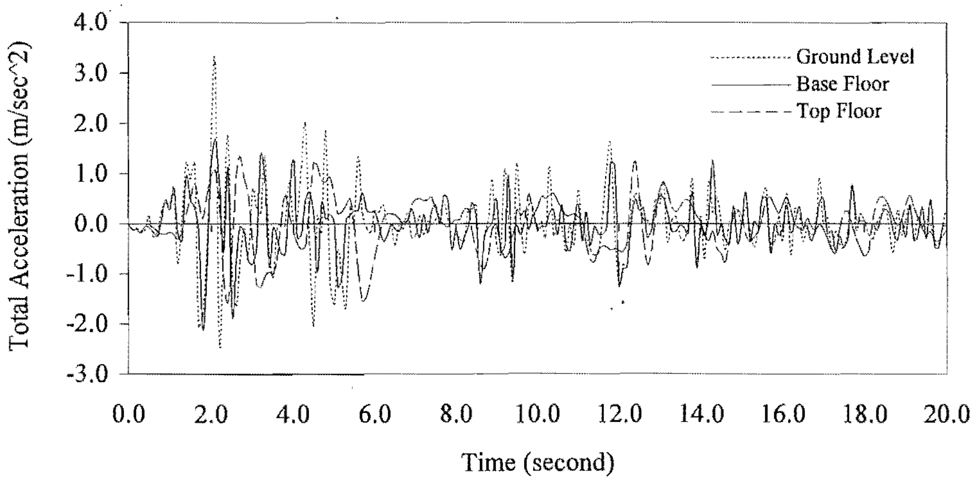


(e) Segmental Building with Foundation compliance

Fig. 6.16 ..... (continued)

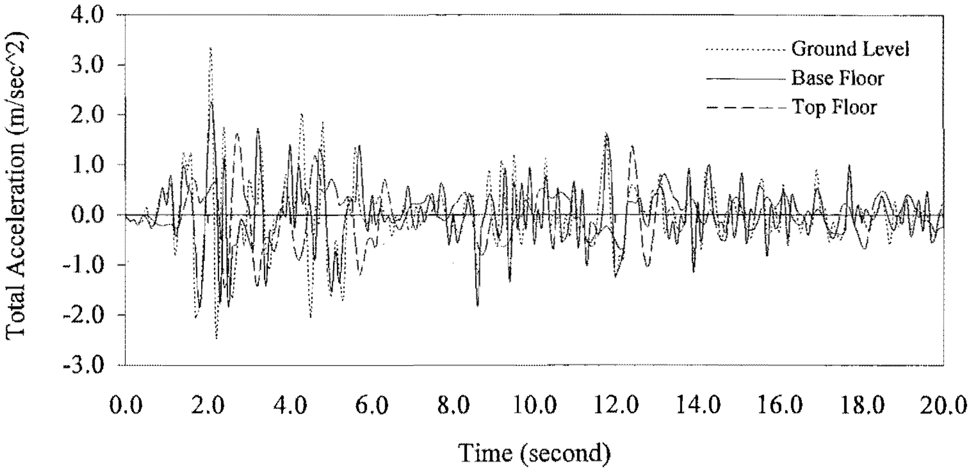


(a) Base Isolated Building with Rigid Base

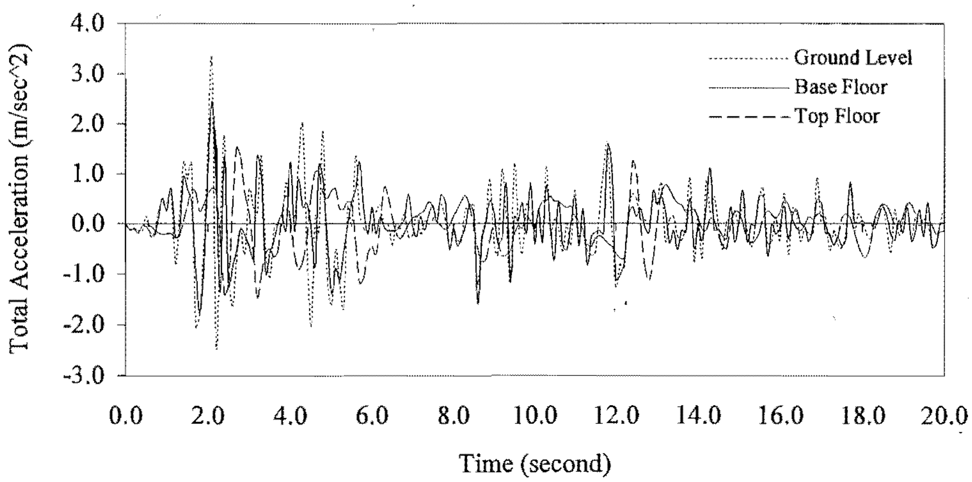


(b) Base Isolated Building with Foundation Compliance

Fig. 6.17 The Response History of Total Accelerations for Structures with Additional Damping Using Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation compliance

Fig. 6.17 ..... (continued)

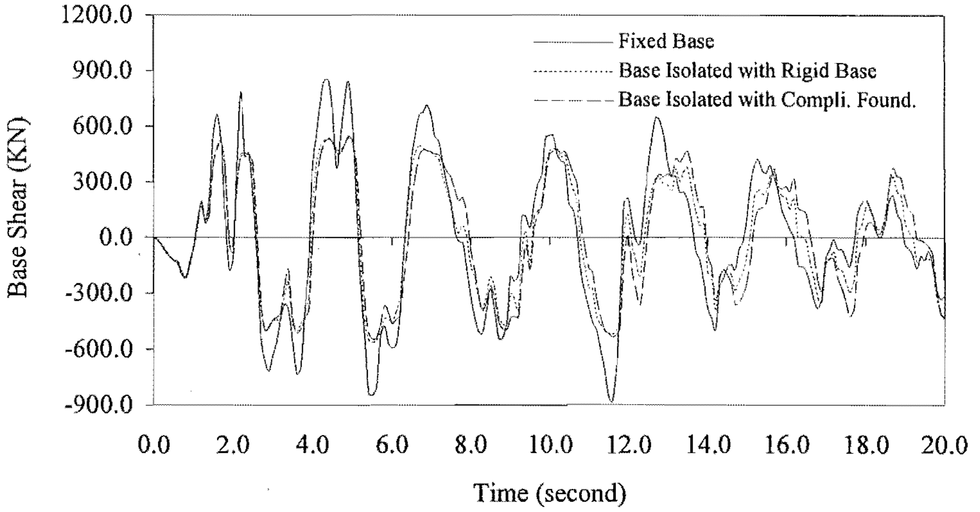
elasto-plastic isolation systems are approximately 10%, 21%, 24%, 10% and 10% less than those without additional damping as shown in Fig. 5.10. From the results for the structures with bilinear isolation devices as shown in Fig. 6.19, the base isolated and segmental buildings with additional damping show a very small difference (less than 7%) in base shears when compared with the same buildings without additional damping as shown in Fig. 5.11.

From the results for the buildings with additional damping using elasto-plastic isolation devices as shown in Fig. 6.18, the base shear responses of the base isolated and segmental structures are smaller than those for the fixed base buildings. Similar responses are observed in the structures with additional damping using bilinear isolation systems as shown in Fig. 6.19.

Based on the time history analyses shown in Table 6.5, the base shears of the base isolated and segmental buildings with additional damping mounted on elasto-plastic and bilinear isolation systems are respectively 43%, 16%, 45% and 35% smaller than those of the fixed base structures with additional damping. For the structures designed to NZS 3101:1982, the base shears of the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolation devices are reduced by approximately 28%, 13%, 31% and 27% when compared with the fixed base structures with additional damping as shown in Table B.2 of Appendix B.

Also, compared with the equivalent static method of NZS 4203:1992 shown in Table 6.5, the structures with additional damping mounted on elasto-plastic and bilinear isolation systems using the time history analyses have smaller base shears except for the fixed base and base isolated building with bilinear isolation devices. Similar results are also observed for the structures designed to NZS 3101:1982 as shown in Table B.2 of Appendix B.

For the structures with additional damping mounted on elasto-plastic isolation systems, Fig. 6.20 shows the lateral storey shear envelopes of the base isolated, fixed base and segmental structures based on the time history analyses and compares them with the equivalent static method of NZS 4203:1992. From the shear diagrams for the base isolated, fixed base and segmental structures, there are reductions of the storey shears over the height except in the lower storeys. Similar results are also obtained in the buildings with additional damping using bilinear isolation systems as shown in Fig. 6.21.



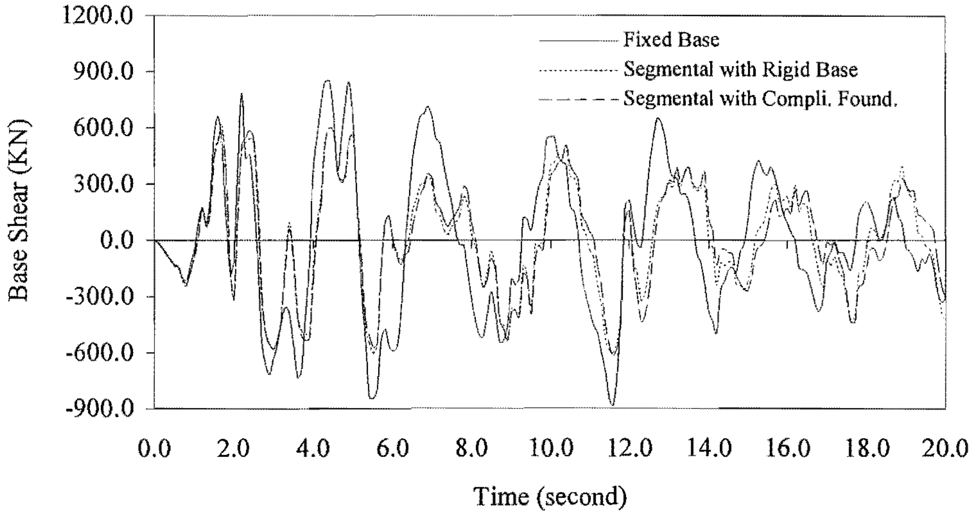
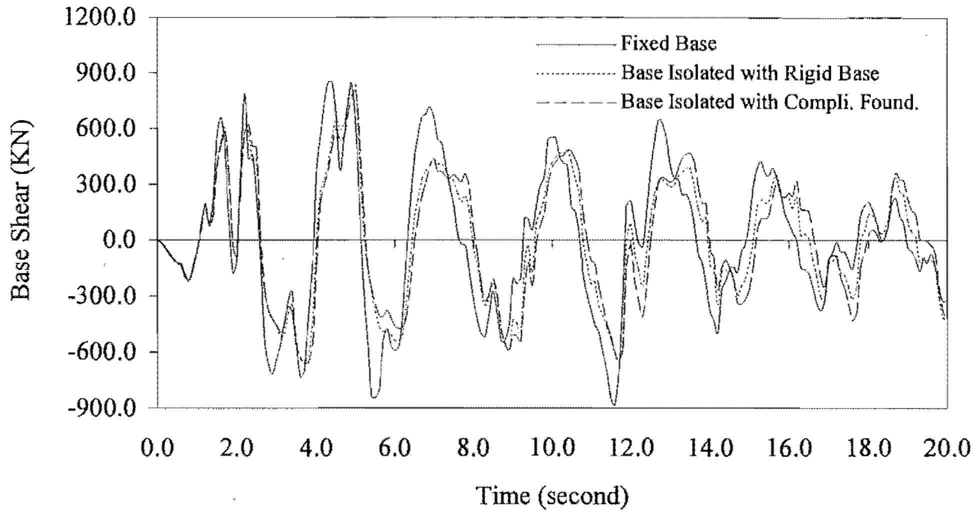


Fig. 6.18 The Response History of Base Shears for Structures with Additional Damping Using Elasto-Plastic Isolation Systems



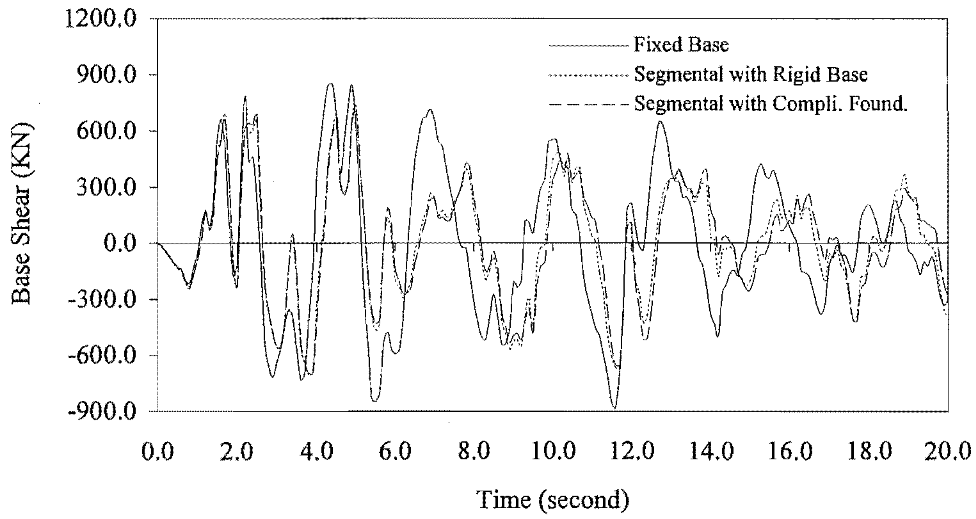
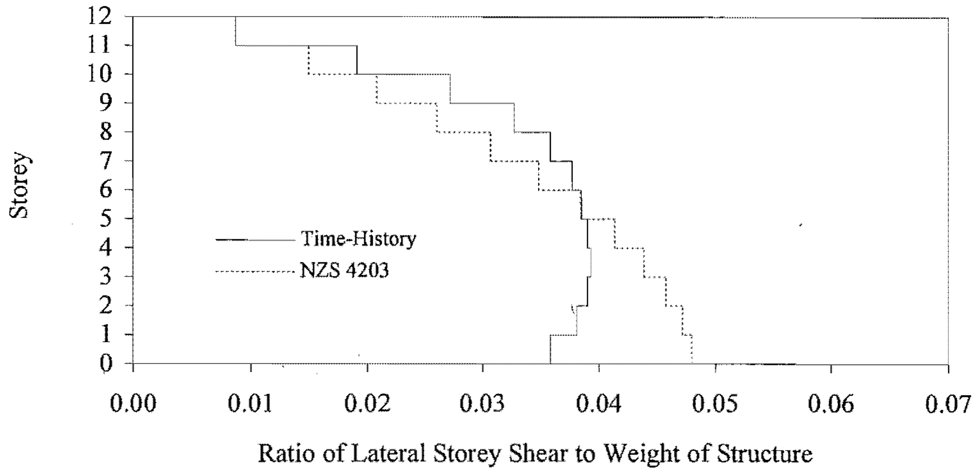


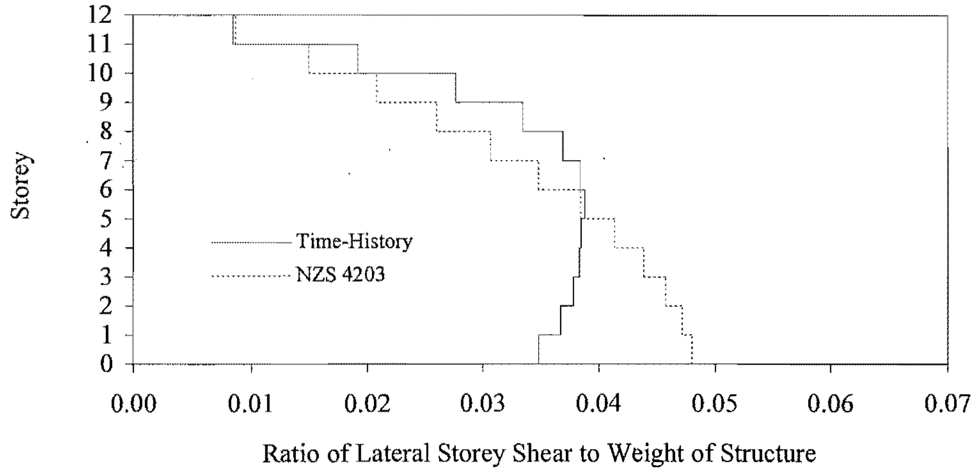
Fig. 6.19 The Response History of Base Shears for Structures with Additional Damping Using Bilinear Isolation Systems

Type	Base Shear / Total Weight of Structure		
	Time History Analysis		Equivalent Static Method of NZS 4203:1992
	Elasto-Plastic Model	Bilinear Model	
Fixed Base Building	0.0607		0.0480
Base Isolated Building on a Rigid Base	0.0358	0.0531	0.0480
Base Isolated Building on a Compliant Foundation	0.0348	0.0510	0.0480
Segmental Building on a Rigid Base	0.0336	0.0404	0.0480
Segmental Building on a Compliant Foundation	0.0338	0.0392	0.0480

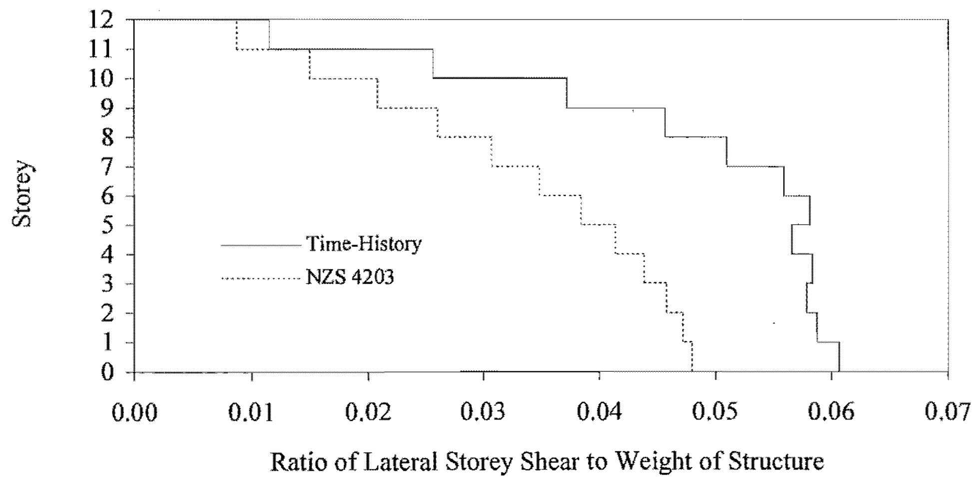
Table 6.5 Normalised Base Shears for Different Types of Buildings with Additional Damping



(a) Base Isolated Building with Rigid Base

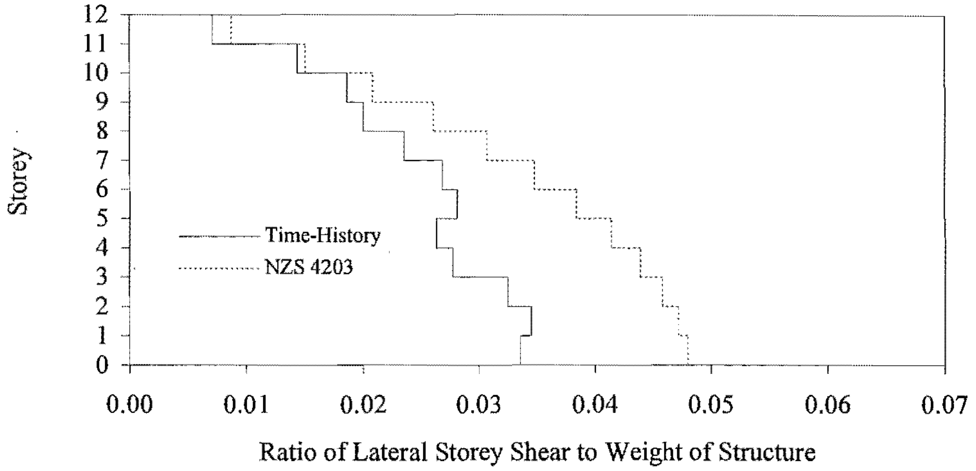


(b) Base Isolated Building with Foundation Compliance

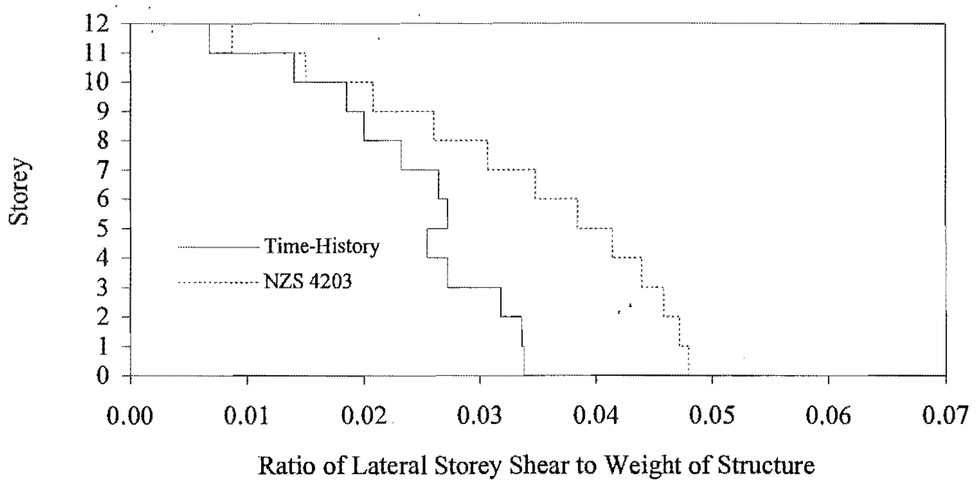


(c) Fixed Base Building

Fig. 6.20 Lateral Storey Shear Envelopes for Structures with Additional Damping Using Elasto-Plastic Isolation Systems

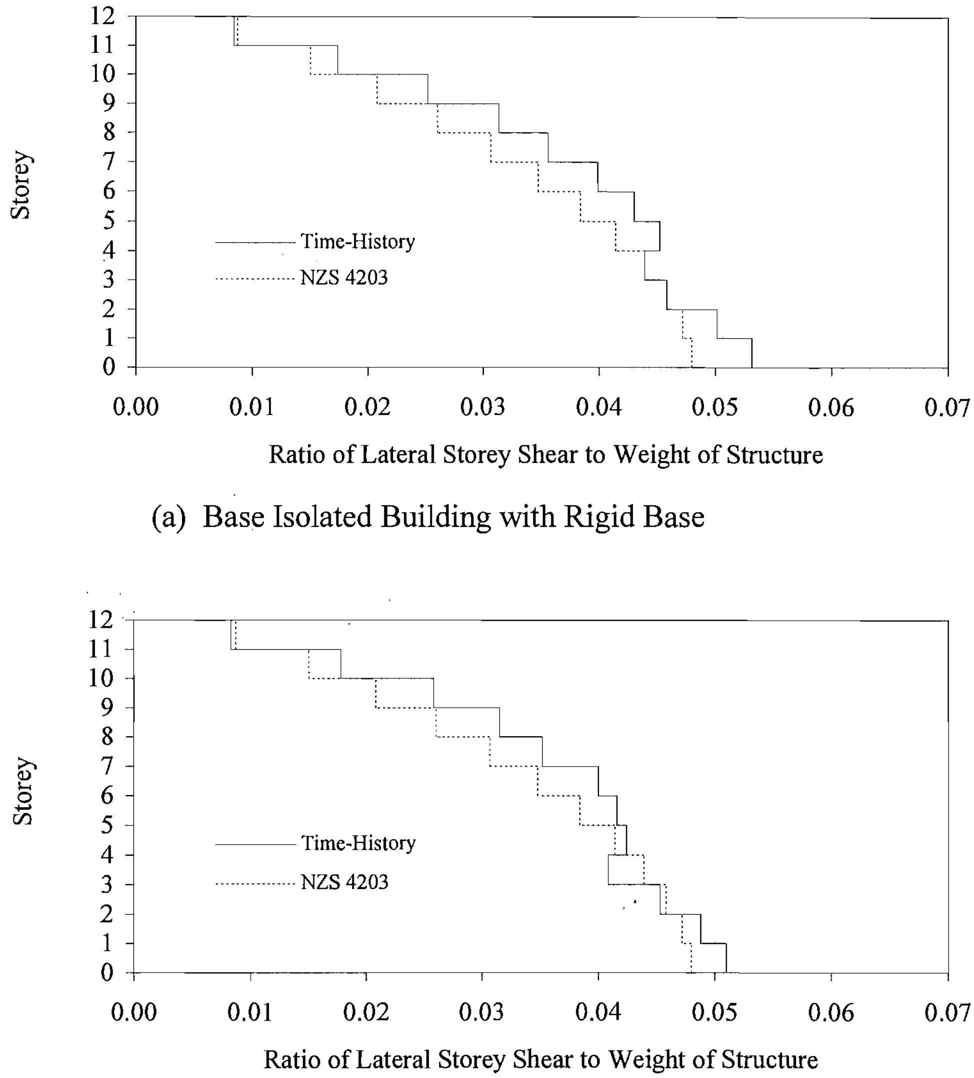


(d) Segmental Building with Rigid Base



(e) Segmental Building with Foundation Compliance

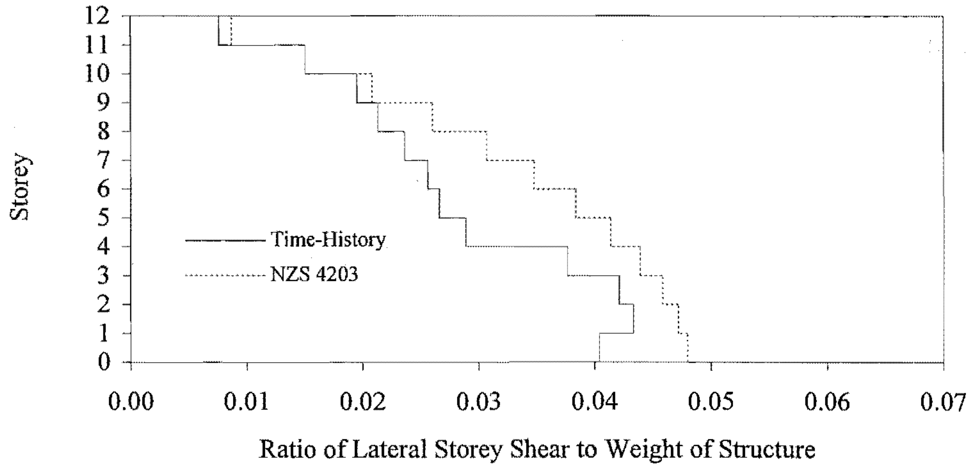
Fig. 6.20 ..... (continued)



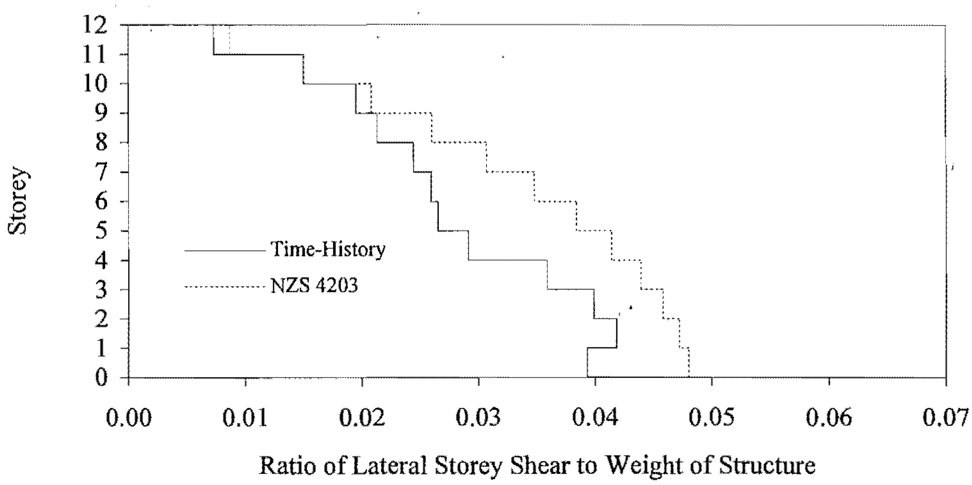
(a) Base Isolated Building with Rigid Base

(b) Base Isolated Building with Foundation Compliance

Fig. 6.21 Lateral Storey Shear Envelopes for Structures with Additional Damping Using Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation Compliance

Fig. 6.21 ..... (continued)

Compared with the time history analyses of Fig. 6.20, the equivalent static lateral force distribution of NZS 4203:1992 gives a safety margin and a smaller estimation of the storey shears at the lower storeys and at the upper storeys for the base isolated buildings with additional damping, and a safety margin of the storey shears over the height for the segmental buildings with additional damping. For the fixed base building with additional damping, there is an underestimation of the storey shears over the whole height of the structure.

When compared with the time history analyses of Fig. 6.21, the equivalent static lateral force distribution of NZS 4203:1992 gives a slight smaller estimation of the storey shears over the height except in the middle storeys for the base isolated with additional damping, and a safety margin of the storey shears over the height for the segmental buildings with additional damping.

For the structures with additional damping using elasto-plastic isolation systems, the lateral storey shear envelopes for the base isolated, fixed base and segmental buildings designed to NZS 3101:1982 are shown in Fig. B.2 of Appendix B. Compared with the time history analyses, the equivalent static lateral force distribution of NZS 4203:1992 gives an underestimation of the storey shears over the height for the fixed base and base isolated buildings except in the lower storeys for the base isolated building, and a safety margin for the storey shears at the lower storeys and a smaller estimation of the storey shears at the upper storeys for the segmental buildings. Similar results are also seen in the structures with additional damping using bilinear isolation devices as shown in Fig. B.3 of Appendix B. This means that the El Centro 1940 N-S excitation is more severe than the design earthquake implied in NZS 4203:1992 at the natural frequencies of these structures.

6.4 Curvature Ductility Demands of Beams and Columns

According to the capacity design method [P2,P5] for the strong column and weak beam type of failure mechanism, the member ductility demand is the most common damage parameter used to show failure or not of members. Therefore, the member damage index depends on the ultimate curvature of the members and it is very important to evaluate the accurate ultimate curvature for the designed members.

As mentioned in the previous chapter, the maximum curvature ductility factors of 20 and 30 for columns and beams are also used here. The curvature ductility demands at the beam ends and column bases for the various types of structures with additional damping subjected to the El Centro 1940 N-S earthquake will be investigated. The 12-storey moment-resistant frame models were used as discussed in Sections 3.4.3 and were designed to NZS 3101:1995.

For the structures with additional damping using elasto-plastic isolation systems shown in Table 6.6 (a), the curvature ductility demands at the internal and external column bases are 1.92 and 1.97 for the fixed base building, less than 1.0 for the base isolated and segmental buildings with either a rigid base or a compliant foundation. Similar results are obtained for the base isolated and segmental buildings with additional damping using bilinear isolation devices as shown in Table 6.6 (b). As mentioned in the previous chapter, the curvature ductility demand of less than 1.0 in the table means that the member remains elastic, i.e. no plastic hinge occurs.

Table 6.6 (a) shows the maximum beam curvature ductility demand is 6.96 for the fixed base building with additional damping. For the structures with additional damping mounted on elasto-plastic isolation systems, the base isolated and segmental buildings with a rigid base and a compliant foundation are 1.04, 1.36, less than 1.0 and 1.24 for the maximum beam curvature ductility demands. These show that the base isolated and segmental structures with additional damping using elasto-plastic isolators have a significant reduction (at least 80% or greater) of the maximum beam curvature ductility demands when compared with those of the fixed base buildings with additional damping. Table 6.6(b) shows very similar results for the base isolated and segmental structures with additional damping using bilinear isolation devices.

As shown in Table 6.6 (a) and compared with values in Table 5.6 (a), the curvature ductility demands at the column bases for the fixed base building with additional damping are smaller than those without additional damping. Similar results also occur with the maximum beam curvature ductility demand for the fixed base building. For the maximum beam curvature ductility demands, the base isolated and segmental buildings with additional damping mounted on elasto-plastic isolation systems are approximately 48% and 56% smaller than those of frames without additional damping. Similar results are observed for the base isolated and segmental structures with additional damping mounted on bilinear isolation devices as shown in Table 6.6 (b) when compared with values in Table 5.6 (b).

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.29	< 1.00	< 1.00	< 1.00	< 1.00
	10	2.06	< 1.00	< 1.00	< 1.00	1.03
	9	2.99	1.04	1.36	< 1.00	1.24
	8	2.89	< 1.00	< 1.00	< 1.00	< 1.00
	7	3.79	< 1.00	< 1.00	< 1.00	< 1.00
	6	4.97	< 1.00	< 1.00	< 1.00	< 1.00
	5	5.43	< 1.00	< 1.00	< 1.00	< 1.00
	4	6.28	< 1.00	< 1.00	< 1.00	< 1.00
	3	6.76	< 1.00	< 1.00	< 1.00	< 1.00
	2	6.96	< 1.00	< 1.00	< 1.00	< 1.00
	1	6.78	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	1.96	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	1.92	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.97	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.29	< 1.00	< 1.00	< 1.00	1.15
	10	2.06	< 1.00	< 1.00	< 1.00	1.14
	9	2.99	< 1.00	1.31	< 1.00	1.37
	8	2.89	< 1.00	< 1.00	< 1.00	< 1.00
	7	3.79	< 1.00	1.22	< 1.00	< 1.00
	6	4.97	< 1.00	< 1.00	< 1.00	< 1.00
	5	5.43	< 1.00	1.02	< 1.00	< 1.00
	4	6.28	< 1.00	1.02	< 1.00	< 1.00
	3	6.76	< 1.00	1.22	< 1.00	1.17
	2	6.96	1.09	1.39	1.10	1.31
	1	6.78	1.02	1.16	< 1.00	1.08
Column Bases	L.Ext.*	1.96	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	1.92	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.97	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:
* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 6.6 Maximum Curvature Ductility Demands for Different Structures with Additional Damping

For the structures designed to NZS 3101:1982 shown in Table B.3 of Appendix B, the maximum beam curvature ductility demands for the base isolated and segmental building with additional damping using elasto-plastic and bilinear isolators are much smaller than those of the fixed base buildings. When compared with values in Table A.3 of Appendix A, the maximum beam curvature ductility demand for the fixed base building with additional damping is one third smaller than that of frame without additional damping, and the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolation devices are over one half smaller than those of ones without additional damping.

6.5 Summary and Conclusion

The application of velocity-dependent damping devices installed at the interior columns of each floor of the structures was used to investigate whether they can be effective in reducing structural response to seismic excitation. A series of time history analyses were carried out for the base isolated multistorey buildings with additional damping using elasto-plastic and bilinear isolation systems and compared with other types of structures such as fixed base and segmental buildings when subjected to N-S component of El Centro 1940 earthquake.

In Section 6.2, additional equivalent viscous damping was computed based on a harmonic loading time history and the maximum applied load was according to equivalent static method of NZS 4203:1992. The effective fundamental period of the base isolated structures can then be measured from the time history plots and compared with the effective period calculated from free-vibration modal analyses.

Using the method mentioned in Section 6.2.4, the calculated total equivalent viscous damping of the 12-storey buildings, with $T_{1\,(UI)}$ from 0.2 to 2.0 seconds, were then compared with the critical damping measured from the displacement spectrum of El Centro 1940 N-S earthquake. Also, a 6-storey building on a fixed base with $T_{1\,(UI)}$ varying from 0.2 to 1.2 seconds was used to compare and verify the effective period and damping achieved from a 12-storey building and a good agreement was obtained.

In Section 6.3, the seismic responses obtained by the different types of structures were discussed. The fixed base building with additional damping has greatly reduced top displacement and interstorey drift when compared with the fixed building without additional damping. For the base shears, the fixed base building with additional damping shows a smaller difference when compared with the fixed building without additional damping. Similar results are shown for the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolation systems.

From the time history analyses presented in Section 6.3, the fixed base building shows that there is a reduction of the storey shears over the height except at the middle storeys. This is true for the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolators. This phenomenon occurs because as the superstructure becomes more flexible the higher modes make more significant contributions especially in the mid-height parts of the building.

From Section 6.3, the fixed base, base isolated and segmental buildings with additional damping have significantly reduced total accelerations and interstorey drifts when compared with the same buildings without additional damping. Therefore, it can be seen that the installation of additional damping devices in a structure may improve the seismic resistance of buildings.

From the results mentioned in Section 6.4, it can be seen that the installation of extra damping devices in a structure will dramatically reduce maximum beam curvature ductility demands when compared with the structures without these additional damping devices. Also, the structure with a stiffer superstructure has a greater reduction of maximum beam curvature ductility demand when compared with the structure with a flexible superstructure. This can be seen from reduction of maximum beam curvature ductility demands for the structures designed to NZS 3101:1982 and NZS 3101:1995, because the structures designed to the earlier code were required to be stiffer.

With the inclusion of isolation systems, the base isolated and segmental buildings have significantly reduced top floor displacements, initial forces, interstorey drifts and ductility demands when compared with the fixed base building. In comparing with the interstorey drifts, the segmental buildings have greater reductions than do the base isolated buildings. The much smaller interstorey drifts reduce the risk to the structure and avoid the occurrence of non-structural damage during the earthquake attacks. For the design of special purposes buildings such as hospitals, police stations, television stations and computer centres, a very small interstorey drift may be required during the earthquake attacks. Therefore, a segmental building would be more appropriate than a base isolated building at the design of these long natural period structures. Due to the significantly reduced ductility demands in the superstructure, it makes possible simplification of the structural detailing and other special design requirements when compared with more conventional design approaches. Therefore, a wider choice of architectural forms and structural materials would be available to the designer.

CHAPTER 7

EFFECTS OF DIFFERENT EARTHQUAKES ON THE SEISMIC RESPONSES OF STRUCTURES

7.1 Introduction

The design earthquake is a specification of the seismic ground motion at a site, used for the earthquake-resistant design of a structure [S2]. It is known that the characteristics of earthquake ground excitations strongly affect both structural deformation and energy dissipation. Because of the uncertainties involved in estimating the nature of ground motions that might occur in the future at a building site, Ref. M1 suggested that several earthquake records should be used during the analyses of building structures.

From Section 4.10.1.3 of NZS 4203:1992 [S8], when the numerical time history analysis for the ultimate limit state building response is used, at least three different earthquake records should be employed. Thus, four earthquake records are used in this study. They are El Centro 1940 N-S, Taft 1952 N69W, Parkfield 1966 N65E and Mexico 1985 SCT S00E (at SCT site). Also mentioned in Section 4.10.2 of NZS 4203:1992, the chosen earthquake records should be scaled by a recognized method which will be discussed in Section 7.2.

As discussed in Section 4.6, design earthquake motions for most seismic areas of the world often use the earthquake record, El Centro 1940, as a basis for seismic design criteria. Thus, as discussed earlier in Chapters 5 and 6, a series of inelastic time history analyses have been carried out to investigate in detail the seismic responses of base isolated multistorey structures with and without additional damping and also to compare with other structures such as fixed base and segmental buildings [C13], under the N-S component of El Centro 1940 earthquake.

In order to understand the seismic behaviour of the structures under the effect of the different ground motions, the evaluation of the major structural response quantities such as base and top floor displacements, lateral storey displacements, interstorey drifts and base shears of the structures will be investigated. There are some concerns on the curvature ductility demands of beams and columns for the base isolated buildings and other types of structures under the different earthquakes mentioned above which will also be discussed in this chapter.

7.2 Scaled Earthquake Records

7.2.1 General

In accordance with Section 4.10.2 of NZS 4203:1992, the time history analyses for the ultimate limit state building responses were carried out using scaled earthquake records. Scaling shall be such that over the period range of interest for the structure being analysed, the 5% damped spectrum of the earthquake record does not differ significantly from the design spectrum for the limit state being considered. Therefore, the earthquake records were scaled according to their 5% damped spectra so as to match the design spectrum in Section 4.6.2.9 (b) (ii) of NZS 4203:1992 for the intermediate soil sites.

Four earthquake records were used for the dynamic analyses in this study with each earthquake record having different characteristics. El Centro 1940 N-S has many peaks with a few pulses at the commencement of the earthquake, Taft 1952 N69W has many peaks of a similar magnitude, Parkfield 1966 N65E has a large pulse over a very short time duration, and Mexico 1985 SCT S00E has the strongest excitation at the long periods.

To obtain the strongest excitation at long natural periods, the Mexico 1985 SCT S00E earthquake was used to investigate the seismic performance and determine whether or not there were benefits to be gained from the longer natural period base isolated and segmental structures. The other three shorter natural period earthquake records with different characteristics were chosen to investigate the effect of seismic responses for the longer natural period structures under the different excitations.

7.2.2 Scale Factors Used in This Study

As discussed in Section 5.3 and shown in Table 5.3, the fundamental periods of the structures are different based on two design standards. Therefore, the scale factors were calculated separately for the structures designed to NZS 3101:1995 [S9] and NZS 3101:1982 [N1], and in accordance with the provisions of loading code NZS 4203:1992.

In general, the displacement responses of the structures are mainly affected by the fundamental modal period of the structures. For the long period structures, the inertial responses are affected by the contributions of the higher modes of the structure and a scaling method that takes a weighted contribution of the first three modes of free-vibration are also considered.

As can be seen from Table 5.3 of Chapter 5, the fundamental periods of the fixed base and base isolated structures are very similar. For the structure designed to NZS 3101:1995, the fundamental, second and third modal periods of the fixed base structure are respectively 2.511, 0.834 and 0.472 seconds. The fundamental, second and third modal periods of the segmental structure are respectively 2.700, 0.920 and 0.557 seconds.

For the structure designed to NZS 3101:1982, the fundamental, second and third modal periods of the fixed base structure are respectively 2.012, 0.705 and 0.396 seconds. The fundamental, second and third modal periods of the segmental structure are 2.261, 0.797 and 0.489 seconds respectively. The fundamental period of the segmental building is 0.2 second greater than that for the fixed base building.

Four methods were used to calculate the scale factors as follows: (1) scaled by fundamental periods of the structures; (2) an average of scaled factors for the fundamental periods from two to three second range of the structures; (3) an average of scaled factors for the fundamental, second and third modal periods of the structures; (4) weighted scale factors for the fundamental (weight 2), and second and third (weight 1) modal periods of the structures.

Tables 7.1 and 7.2 show scale factors for the fixed base and base isolated, and segmental buildings designed to NZS 3101:1995 and NZS 3101:1982, using the four methods mentioned above for the four earthquake records, respectively. Before obtaining the appropriate scale factors for the chosen earthquake records, the base shears of the structures were used to decide which method is best. Tables 7.3 and 7.4 show base shears to scale factors for the fixed base and base isolated, and segmental buildings for the four earthquake records, respectively.

Earthquake Records	Scale Factors			
	Method 1	Method 2	Method 3	Method 4
	T_1	$T_{1\,(2\text{-}3)}$	$(T_1+T_2+T_3)/3$	$(2T_1+T_2+T_3)/4$
El Centro 1940 N-S	1.13	1.27	1.05	1.07
Taft 1952 N69W	3.59	3.36	2.72	2.94
Parkfield 1966 N65E	0.80	0.81	0.69	0.72
Mexico 1985 SCT S00E	0.61	0.59	3.40	2.70

(a) Structures Designed to NZS 3101:1995

Earthquake Records	Scale Factors			
	Method 1	Method 2	Method 3	Method 4
	T_1	$T_{1\,(2\text{-}3)}$	$(T_1+T_2+T_3)/3$	$(2T_1+T_2+T_3)/4$
El Centro 1940 N-S	1.41	1.27	1.25	1.29
Taft 1952 N69W	2.81	3.36	2.42	2.52
Parkfield 1966 N65E	0.68	0.81	0.55	0.59
Mexico 1985 SCT S00E	0.42	0.59	3.56	2.78

(b) Structures Designed to NZS 3101:1982

Note:

T_1, T_2 and T_3 are each scale factor for the fundamental, second and third modal period of the structure respectively.

$T_{1\,(2\text{-}3)}$ is a average of scale factors for the fundamental periods from two to three second range of the structures.

Table 7.1 Scale Factors for Fixed Base and Base Isolated Buildings under the Four Earthquake Records

Earthquake Records	Scale Factors			
	Method 1	Method 2	Method 3	Method 4
	T_1	$T_{1\,(2\text{-}3)}$	$(T_1+T_2+T_3)/3$	$(2T_1+T_2+T_3)/4$
El Centro 1940 N-S	1.30	1.27	1.05	1.12
Taft 1952 N69W	3.72	3.36	2.83	3.05
Parkfield 1966 N65E	0.85	0.81	0.77	0.79
Mexico 1985 SCT S00E	0.62	0.59	2.78	2.24

(a) Structures Designed to NZS 3101:1995

Earthquake Records	Scale Factors			
	Method 1	Method 2	Method 3	Method 4
	T_1	$T_{1\,(2\text{-}3)}$	$(T_1+T_2+T_3)/3$	$(2T_1+T_2+T_3)/4$
El Centro 1940 N-S	1.15	1.27	1.06	1.08
Taft 1952 N69W	3.24	3.36	2.61	2.76
Parkfield 1966 N65E	0.78	0.81	0.68	0.70
Mexico 1985 SCT S00E	0.52	0.59	3.37	2.66

(b) Structures Designed to NZS 3101:1982

Note:

T_1, T_2 and T_3 are each scale factor for the fundamental , second and third modal period of the structure respectively.

$T_{1\,(2\text{-}3)}$ is a average of scale factors for the fundamental periods from two to three second range of the structures.

Table 7.2 Scale Factors for Segmental Buildings under the Four Earthquake Records

From Tables 7.3 and 7.4, method 4 is a better approach for the structures designed to NZS 3101:1995 and NZS 3101:1982 because the base shears obtained have the smallest difference under the four earthquake records. In this method, a very large top floor deflection (1.42 m) of the structure during the time history analyses was obtained for the Mexico 1985 SCT S00E earthquake. This exceeds the 2.5% interstorey deflection limit specified in Section 2.5.4.5 (b) of NZS 4203:1992 when time history analyses are used. Compared with other methods, the scale factor for method 1 is smaller than and closer to that of method 4. Therefore, method 1 was used to compute the scale factor for the Mexico 1985 SCT S00E earthquake.

Finally, the scale factors are summarised in Table 7.5 for the fixed base, base isolated and segmental structures designed to NZS 3101:1995 and NZS 3101:1982 respectively. The base shear coefficient for each earthquake response spectrum and the design spectrum of NZS 4203:1992 for the intermediate soil sites for the fixed base and base isolated, and segmental structures designed to NZS 3101:1995 and NZS 3101:1982 are shown in Figs. C.1, C.2, C.3 and C.4 of Appendix C, respectively.

7.3 Overall Response Quantities of Structures

7.3.1 General

During a strong earthquake, it is expected that displacement-dependent base isolation devices will provide sufficient horizontal flexibility to lengthen the fundamental period of the structures and will supply some extra damping due to their hysteretic damping. For some earthquakes, such as El Centro 1940 N-S, Taft 1952 N69W and Parkfield 1966 N65E, with spectral accelerations which reach their peaks in the short period region and diminish in longer periods, the fundamental period shift by base isolation systems will definitely reduce the earthquake energy transmitted to the structure.

It must also be recognised that occasionally earthquakes give their strongest excitation at long periods. The Mexico 1985 SCT S00E earthquake is the best known example. With this type of motion which has peak spectral acceleration in long period region, shifting the fundamental period of the structure by base isolation systems is not beneficial as it may place the structure in a more dominant earthquake energy region.

Earthquake Records	Method	Scale Factors	Base Shear / Total Weight of Building
El Centro 1940 N-S	1	1.13	0.0711
	2	1.27	0.0723
	3	1.05	0.0680
	4	1.07	0.0689
Taft 1952 N69W	1	3.59	0.0797
	2	3.36	0.0786
	3	2.72	0.0710
	4	2.94	0.0748
Parkfield 1966 N65E	1	0.80	0.0752
	2	0.81	0.0756
	3	0.69	0.0701
	4	0.72	0.0716
Mexico 1985 SCT S00E	1	0.62	0.0595
	2	0.59	0.0592
	3	3.07	0.0825
	4	2.46	0.0788

(a) Structures Designed to NZS 3101:1995

Earthquake Records	Method	Scale Factors	Base Shear / Total Weight of Building
El Centro 1940 N-S	1	1.41	0.0846
	2	1.27	0.0821
	3	1.25	0.0817
	4	1.29	0.0826
Taft 1952 N69W	1	2.81	0.0826
	2	3.36	0.0839
	3	2.42	0.0820
	4	2.52	0.0822
Parkfield 1966 N65E	1	0.68	0.0799
	2	0.81	0.0818
	3	0.55	0.0777
	4	0.59	0.0780
Mexico 1985 SCT S00E	1	0.42	0.0748
	2	0.59	0.0763
	3	3.56	0.0969
	4	2.78	0.0899

(b) Structures Designed to NZS 3101:1982

Table 7.3 Base Shears of Fixed Base and Base Isolated Buildings at Different Scale Factors for the Four Earthquake Records

Earthquake Records	Method	Scale Factors	Base Shear / Total Weight of Building
El Centro 1940 N-S	1 2 3 4	1.30 1.27 1.05 1.12	0.0490 0.0475 0.0413 0.0434
Taft 1952 N69W	1 2 3 4	3.72 3.36 2.83 3.05	0.0680 0.0637 0.0562 0.0603
Parkfield 1966 N65E	1 2 3 4	0.85 0.81 0.77 0.79	0.0693 0.0680 0.0665 0.0672
Mexico 1985 SCT S00E	1 2 3 4	0.62 0.59 2.78 2.24	0.0447 0.0441 0.0733 0.0685

(a) Structures Designed to NZS 3101:1995

Earthquake Records	Method	Scale Factors	Base Shear / Total Weight of Building
El Centro 1940 N-S	1 2 3 4	1.15 1.27 1.06 1.08	0.0653 0.0704 0.0617 0.0624
Taft 1952 N69W	1 2 3 4	3.24 3.36 2.61 2.76	0.0751 0.0767 0.0653 0.0669
Parkfield 1966 N65E	1 2 3 4	0.78 0.81 0.68 0.70	0.0770 0.0773 0.0738 0.0747
Mexico 1985 SCT S00E	1 2 3 4	0.52 0.59 3.37 2.66	0.0538 0.0559 0.0850 0.0805

(b) Structures Designed to NZS 3101:1982

Table 7.4 Base Shears of Segmental Buildings at Different Scale Factors for the Four Earthquake Records

Earthquake Records	Scale Factors	
	Fixed Base and Base Isolated Buildings	Segmental Buildings
El Centro 1940 N-S	1.07	1.12
Taft 1952 N69W	2.94	3.05
Parkfield 1966 N65E	0.72	0.79
Mexico 1985 SCT S00E	0.61	0.62

(a) Structures Designed to NZS 3101:1995

Earthquake Records	Scale Factors	
	Fixed Base and Base Isolated Buildings	Segmental Buildings
El Centro 1940 N-S	1.29	1.08
Taft 1952 N69W	2.52	2.76
Parkfield 1966 N65E	0.59	0.70
Mexico 1985 SCT S00E	0.42	0.52

(b) Structures Designed to NZS 3101:1982

Table 7.5 Scale Factors for the Four Earthquake Records

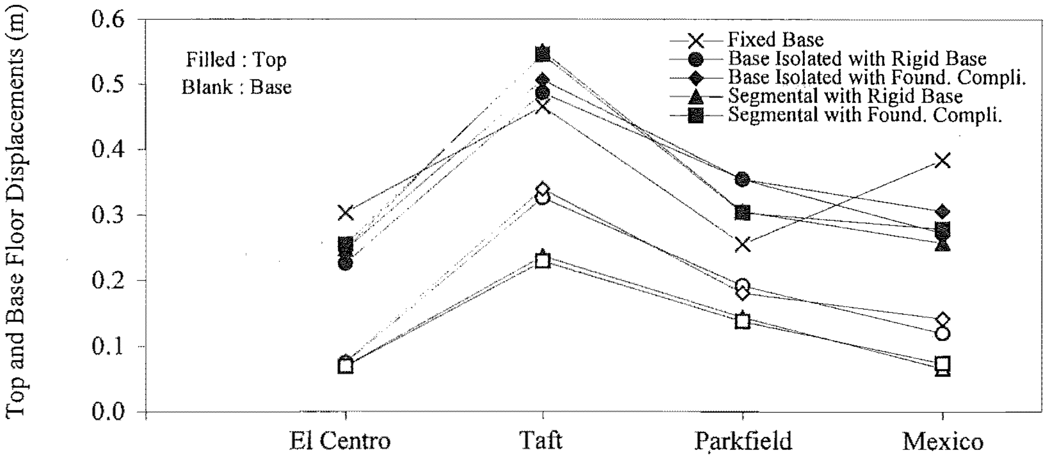
In this section, extensive analyses will be carried out to evaluate the structural responses of the 12-storey structures when subjected to the four scaled earthquakes mentioned above. The frame structure is modelled to deform in a shear-like manner and is designed to NZS 3101:1995 as discussed in Sections 3.4.3 and 3.6. The base isolator has the initial stiffness, k_o, of 10.0 W/m, the post-yield stiffness, αk_o, of 0 and 0.4 W/m, i.e. $\alpha = 0$ and 0.04 for the base isolated structures, and 0 or 0.5 W/m, i.e. $\alpha = 0$ and 0.05 for the segmental structures for the elasto-plastic and bilinear models. The yield strengths, F_y, are respectively 3% and 5%W for the structures designed to NZS 3101:1995 and NZS 3101:1982, where W is the total weight of the structure.

7.3.2 Top and Base Floor Displacements

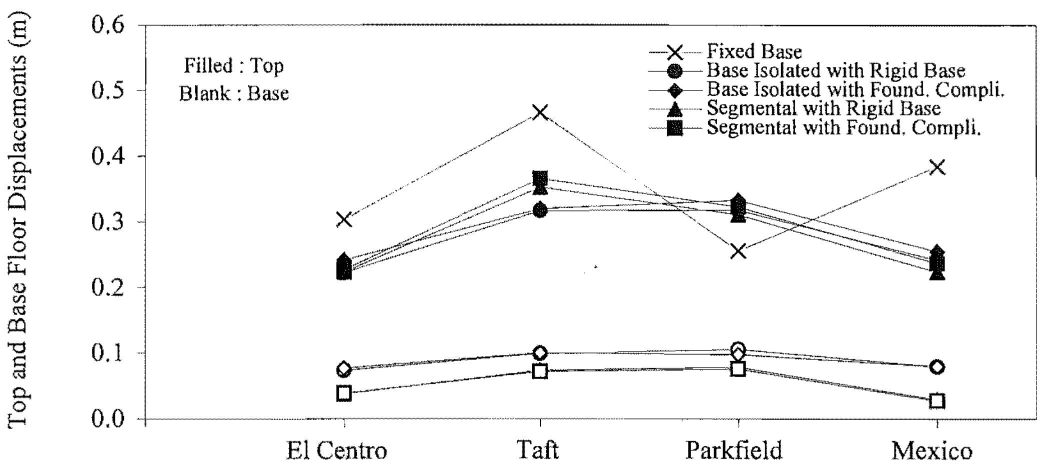
a. Structures without Additional Damping

Fig.7.1 shows the top and base floor displacements of the fixed base, base isolated and segmental buildings with elasto-plastic and bilinear isolation systems when subjected to the El Centro 1940 N-S, Taft 1952 N69W, Parkfield 1966 N65E and Mexico 1985 SCT S00E earthquakes, respectively. It can be seen that the base floor displacements of the base isolated and segmental structures with elasto-plastic isolation systems are greater than those with bilinear isolation devices. This is also true for the top floor displacements for the base isolated and segmental buildings. Similar results are observed in the structures designed to NZS 3101:1982 as shown in Fig. C.5 of Appendix C. This means that the base isolated and segmental buildings using bilinear models increase their additional damping due to the hysteretic behaviour of the isolators more than in the case of the same buildings using elasto-plastic models. Thus, the increased damping reduces lateral displacements for the base isolated and segmental structures.

For the structures with elasto-plastic and bilinear isolation systems, the top floor displacements of the base isolated and segmental structures relative to their base floor displacements are much smaller than the top floor displacements for the fixed base buildings. These results for the structures under the four earthquakes are summarised in Table 7.6. Similar results are also obtained for the structures designed to NZS 3101:1982 as shown in Table C.1 of Appendix C.



(a) Structures with Elasto-Plastic Isolation Systems



(b) Structures with Bilinear Isolation Systems

Fig. 7.1 Top and Base Floor Displacements of the Structures for Different Earthquakes

Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.304	0.466	0.256	0.385
Base Isolated Building with Rigid base	0.151	0.161	0.163	0.153
Base Isolated Building with Foundation Compliance	0.171	0.167	0.174	0.165
Segmental Building with Rigid Base	0.178	0.314	0.162	0.192
Segmental Building with Foundation Compliance	0.186	0.316	0.166	0.205

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.304	0.466	0.256	0.385
Base Isolated Building with Rigid base	0.149	0.218	0.212	0.162
Base Isolated Building with Foundation Compliance	0.165	0.222	0.235	0.175
Segmental Building with Rigid Base	0.186	0.279	0.232	0.194
Segmental Building with Foundation Compliance	0.189	0.294	0.248	0.209

(b) Structures Mounted on Bilinear Isolation Systems

Note:

Top floor displacements of the base isolated and segmental buildings are based on the deduction of base displacements.

Table 7.6 Top Floor Displacements of Structures for Different Earthquakes

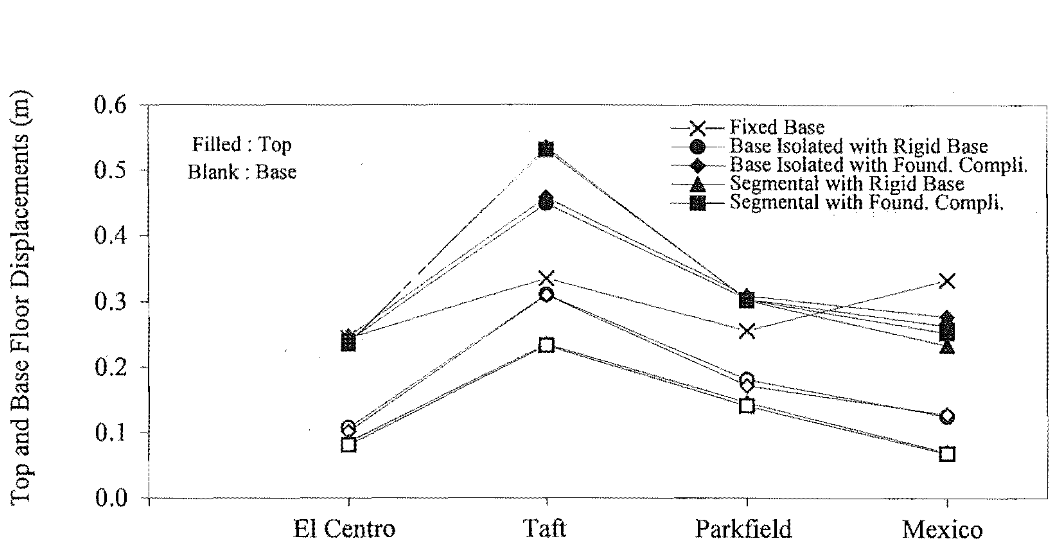
b. Structures with Additional Damping

As shown in Fig. 7.2, the structures with additional damping have reduced displacements when compared with those without additional damping as given in Fig. 7.1. This is true for both the top and base floor displacements of the structures. Similar results are observed in the structures designed to NZS 3101:1982 as shown in Fig. C.6 of Appendix C.

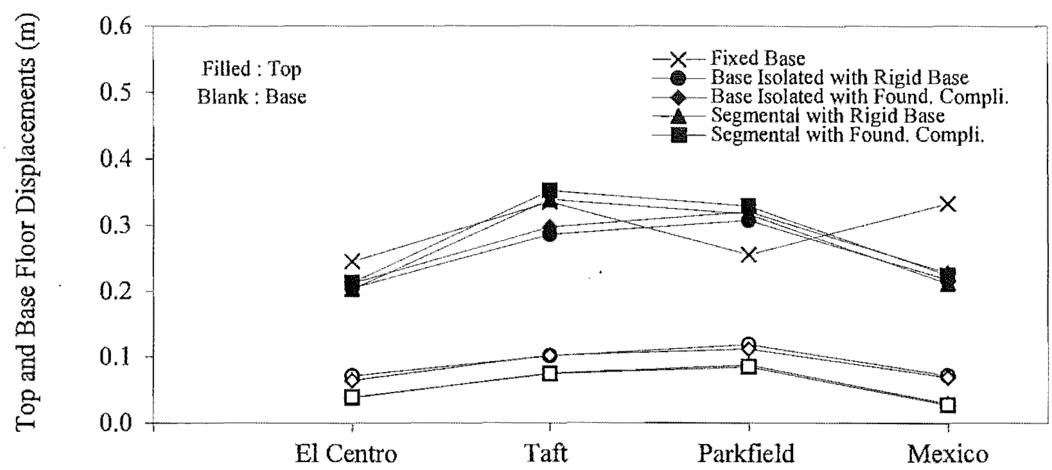
Table 7.7 summarises the top floor displacements for the structures with additional damping under the four earthquakes. It can be found that the structures with additional damping mounted on elasto-plastic isolation devices have greater reduction of displacements than those mounted on bilinear isolation systems. Similar results are also seen for the structures designed to NZS 3101:1982 as shown in Table C.2 of Appendix C.

When compared with Tables 7.6 and 7.7, the top floor displacements of the fixed base buildings with additional damping are approximately 19%, 28%, 3% and 15% less than those without additional damping when subjected to the El Centro 1940 N-S, Taft 1952 N69W, Parkfield 1966 N65E and Mexico 1985 SCT S00E earthquakes. For the structures with elasto-plastic isolation systems, the base isolated and segmental structures with additional damping on a rigid base and a compliant foundation show that the top floor displacements relative to their base floor displacements are reduced by approximately 12%, 15%, 16% and 16% for the El Centro 1940 N-S, 15%, 12%, 5% and 6% for the Taft 1952 N69W, 25%, 21%, 4% and 3% for the Parkfield 1966 N65E and 10%, 10%, 15% and 10% for the Mexico 1985 SCT S00E earthquakes when compared with the same buildings without additional damping.

For the structures with bilinear isolation systems shown in Tables 7.6 and 7.7, the top floor displacements relative to their base floor displacements for the base isolated and segmental structures with additional damping on a rigid base and a compliant foundation are reduced by approximately 10%, 10%, 11% and 8% for the El Centro 1940 N-S, 15%, 12%, 5% and 6% for the Taft 1952 N69W, 11%, 11%, 2% and 2% for the Parkfield 1966 N65E and 10%, 9%, 6% and 5% for the Mexico 1985 SCT S00E earthquakes when compared with the same buildings without additional damping.



(a) Structures with Elasto-Plastic Isolation Systems



(b) Structures with Bilinear Isolation Systems

Fig. 7.2 Top and Base Floor Displacements of the Structures with Additional Damping for Different Earthquakes

Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.245	0.335	0.248	0.333
Base Isolated Building with Rigid base	0.133	0.138	0.123	0.138
Base Isolated Building with Foundation Compliance	0.146	0.148	0.137	0.149
Segmental Building with Rigid Base	0.150	0.300	0.156	0.163
Segmental Building with Foundation Compliance	0.157	0.298	0.162	0.184

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.245	0.335	0.248	0.333
Base Isolated Building with Rigid base	0.134	0.185	0.189	0.146
Base Isolated Building with Foundation Compliance	0.148	0.196	0.209	0.159
Segmental Building with Rigid Base	0.165	0.264	0.229	0.183
Segmental Building with Foundation Compliance	0.174	0.278	0.244	0.198

(b) Structures Mounted on Bilinear Isolation Systems

Note:

Top floor displacements of the base isolated and segmental buildings are based on the deduction of base displacements.

Table 7.7 Top Floor Displacements of Structures with Additional Damping for Different Earthquakes

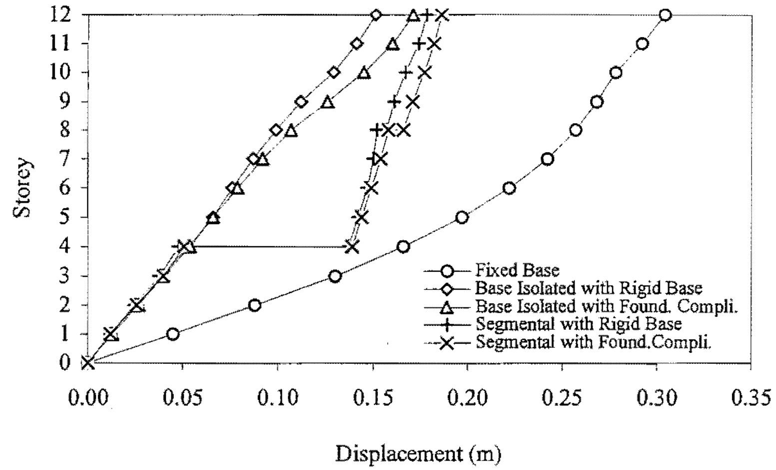
From the results obtained above, the top floor displacement of the fixed base building with additional damping has a slight reduction when compared with the building without additional damping for the Parkfield 1966 N65E earthquake. Similar results are also seen for the segmental buildings with additional damping using elasto-plastic and bilinear isolators for the Taft 1952 N69W and Parkfield 1966 N65E, and the segmental buildings with additional damping with bilinear isolation devices for the Mexico 1985 SCT S00E earthquakes.

7.3.3 Lateral Storey Displacements and Interstorey Drifts

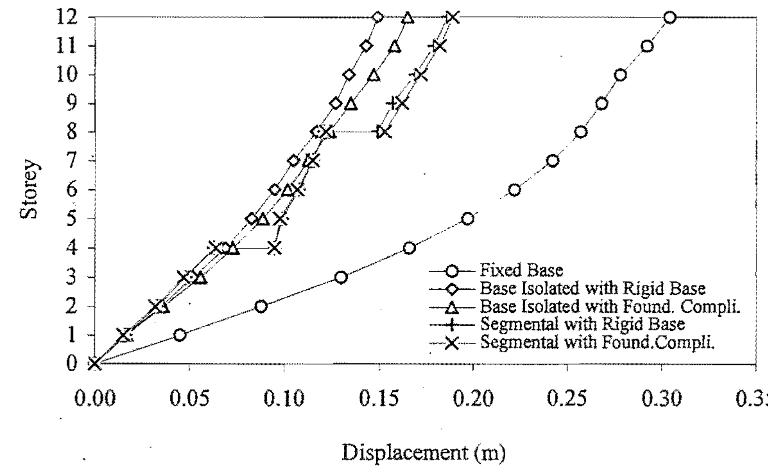
When compared with the fixed base building, the base isolated and segmental structures have dramatically reduced storey displacements as shown in Fig. 7.3 (a) and (b) and actually have much smaller interstorey drifts as shown in Table 7.8. This is true for both the elasto-plastic and bilinear isolation devices when subjected to the El Centro 1940 N-S earthquake. This is similar to results obtained for the structures with additional damping mounted on elasto-plastic and bilinear isolators as shown in Fig. 7.3 (c), (d) and Table 7.9. Similar results are also seen for the structures with and without additional damping using elasto-plastic and bilinear isolation systems based on NZS 3101:1982 as shown in Fig. C.7, Tables C.3 and C.4 of Appendix C.

For the structures with elasto-plastic and bilinear isolation devices, the base isolated and segmental buildings have significantly reduced storey displacements over the whole building height when compared with the fixed base building for the Taft 1952 N69W earthquake as shown in Fig. 7.4 (a) and (b). From the figure, there are greater interstorey deflections between the lower two segments of the segmental buildings using elasto-plastic and bilinear isolation systems, and the former structures exceed the 2.5% limit as specified by NZS 4203:1992 for time history analyses. Similar results are also seen for the structures with additional damping having elasto-plastic and bilinear isolation systems as shown in Fig. 7.4 (c) and (d).

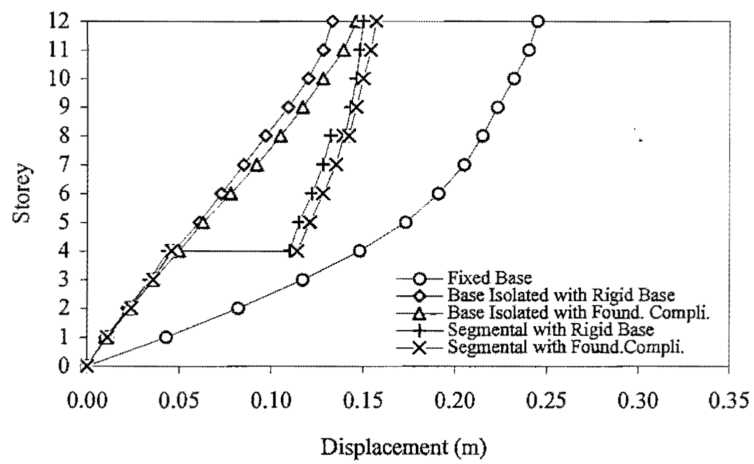
For the structures designed to NZS 3101:1982 under the Taft 1952 N69W earthquake, the base isolated and segmental buildings using elasto-plastic and bilinear isolators have reduced storey displacements when compared with the fixed base building as shown in Fig. C.8 (a) and (b) of Appendix C. However, the interstorey deflections between the lower two segments of the segmental buildings using elasto-plastic isolation systems exceed the 2.5% drift limit. For the structures with extra damping devices, the base isolated and segmental buildings have greatly



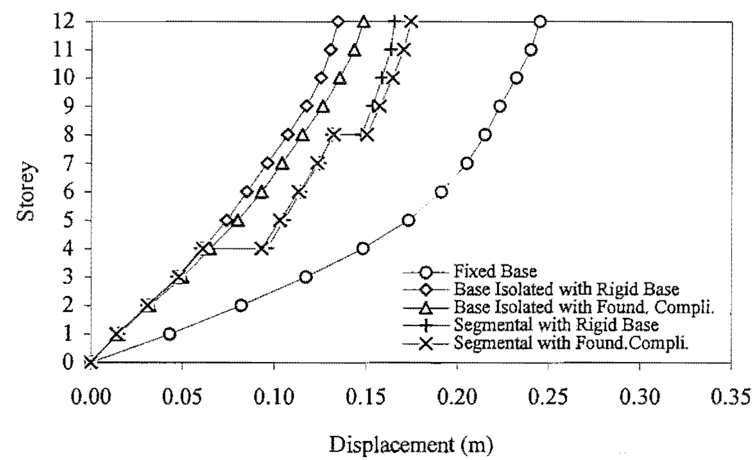
(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems



(c) Structures with Additional Damping Mounted on Elasto-Plastic Isolation Systems



(d) Structures with Additional Damping Mounted on Bilinear Isolation Systems

Fig. 7.3 Comparisons of Displacement with Storey of Different Structures for the El Centro 1940 N-S Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.27%	0.30%	0.11%	0.11%
11	0.38%	0.33%	0.41%	0.19%	0.14%
10	0.27%	0.44%	0.52%	0.16%	0.16%
9	0.30%	0.36%	0.52%	0.11%	0.14%
8	0.41%	0.33%	0.41%	0.08%	0.11%
7	0.55%	0.30%	0.36%	0.08%	0.14%
6	0.68%	0.27%	0.36%	0.14%	0.14%
5	0.85%	0.36%	0.33%	0.14%	0.14%
4	0.99%	0.38%	0.38%	0.27%	0.30%
3	1.15%	0.36%	0.36%	0.36%	0.38%
2	1.18%	0.38%	0.38%	0.36%	0.38%
1	0.90%	0.24%	0.26%	0.24%	0.24%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.16%	0.19%	0.19%	0.19%
11	0.38%	0.25%	0.30%	0.27%	0.27%
10	0.27%	0.19%	0.33%	0.33%	0.27%
9	0.30%	0.27%	0.30%	0.22%	0.25%
8	0.41%	0.33%	0.30%	0.16%	0.19%
7	0.55%	0.27%	0.30%	0.19%	0.22%
6	0.68%	0.33%	0.36%	0.25%	0.25%
5	0.85%	0.38%	0.44%	0.14%	0.14%
4	0.99%	0.47%	0.47%	0.41%	0.47%
3	1.15%	0.47%	0.55%	0.41%	0.41%
2	1.18%	0.52%	0.52%	0.47%	0.47%
1	0.90%	0.32%	0.34%	0.32%	0.30%

(b) Structures Mounted on Bilinear Isolation Systems

Table 7.8 Interstorey Drifts of Structures for the El Centro 1940 N-S Earthquake

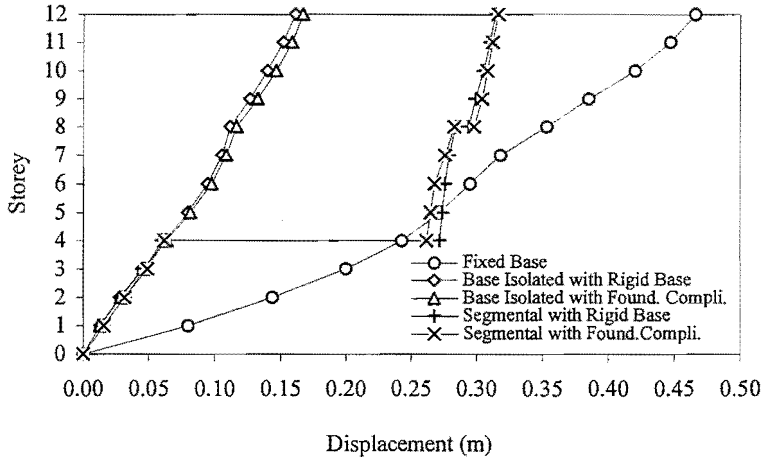
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.14%	0.19%	0.08%	0.08%
11	0.22%	0.22%	0.30%	0.08%	0.11%
10	0.25%	0.30%	0.30%	0.08%	0.11%
9	0.22%	0.33%	0.33%	0.08%	0.11%
8	0.27%	0.33%	0.36%	0.11%	0.11%
7	0.38%	0.33%	0.38%	0.16%	0.19%
6	0.49%	0.33%	0.41%	0.19%	0.19%
5	0.68%	0.38%	0.36%	0.14%	0.19%
4	0.85%	0.36%	0.38%	0.27%	0.27%
3	0.96%	0.33%	0.36%	0.30%	0.33%
2	1.07%	0.33%	0.36%	0.33%	0.36%
1	0.86%	0.20%	0.20%	0.22%	0.22%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

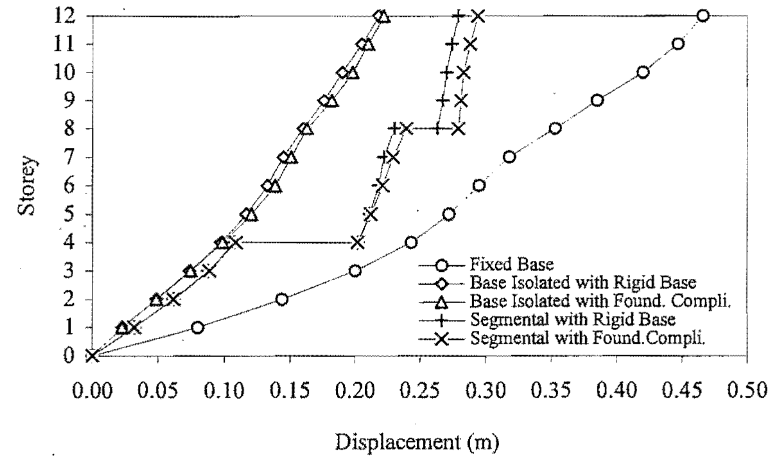
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.11%	0.14%	0.08%	0.11%
11	0.22%	0.14%	0.22%	0.14%	0.16%
10	0.25%	0.22%	0.25%	0.14%	0.19%
9	0.22%	0.27%	0.30%	0.16%	0.19%
8	0.27%	0.30%	0.30%	0.19%	0.25%
7	0.38%	0.30%	0.30%	0.27%	0.27%
6	0.49%	0.30%	0.36%	0.25%	0.27%
5	0.68%	0.36%	0.41%	0.25%	0.27%
4	0.85%	0.38%	0.41%	0.36%	0.36%
3	0.96%	0.44%	0.49%	0.44%	0.47%
2	1.07%	0.44%	0.47%	0.44%	0.47%
1	0.86%	0.30%	0.30%	0.30%	0.28%

(b) Structures Mounted on Bilinear Isolation Systems

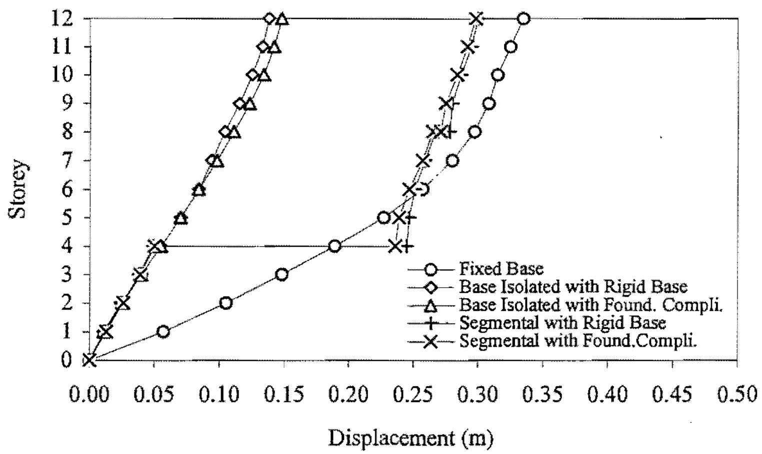
Table 7.9 Interstorey Drifts of Structures with Additional Damping for the El Centro 1940 N-S Earthquake



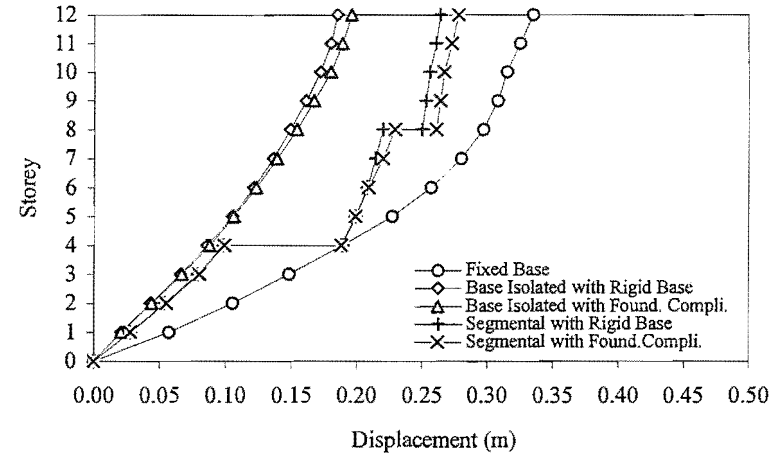
(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems



(c) Structures with Additional Damping Mounted on Elasto-Plastic Isolation Systems



(d) Structures with Additional Damping Mounted on Bilinear Isolation Systems

Fig. 7.4 Comparisons of Displacement with Storey of Different Structures for the Taft 1952 N69W Earthquake reduced storey displacements, especially the interstorey deflections between the lower two segments for the segmental buildings using elasto-plastic isolation devices which are less than the 2.5% limit for the interstorey deflections as shown in Fig. C.8 (c) and (d).

From Table 7.10, interstorey drifts of over 1.7% occurred at the lower storeys of the fixed base building, but the base isolated and segmental buildings with elasto-plastic and bilinear isolation systems show less than 1.0% interstorey drifts under the Taft 1952 N69W earthquake. Compared with Table 7.10, the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolation systems have significantly reduced interstorey drifts, and the fixed base building with additional damping has also reduced interstorey drifts of 1.3% at the lower storeys as shown in Table 7.11.

For the structures designed to NZS 3101:1982 under the Taft 1952 N69W earthquake, the base isolated and segmental buildings using elasto-plastic and bilinear isolation devices have greatly reduced interstorey drifts when compared with the fixed base building as shown in Table C.5 of Appendix C. Similar results are also obtained for the structures with additional damping using elasto-plastic and bilinear isolation systems as shown in Table C.6 of Appendix C.

As shown in Fig. 7.5 (a) and (b), the base isolated and segmental structures using elasto-plastic isolation systems show smaller storey displacements than those with bilinear isolation systems and they have smaller storey displacements than the fixed base building for the Parkfield 1966 N65E earthquake. From the figure, the segmental structures with bilinear isolators show greater interstorey deflections between the lower two segments, but they do not exceed 2.5% interstorey deflection limit as specified by NZS 4203:1992. This is similar to results obtained for the structures with additional damping using elasto-plastic and bilinear isolators under the Parkfield 1966 N65E earthquake as shown in Fig. 7.5 (c) and (d). Similar results are also seen for the structures designed to NZS 3101:1982 as shown in Fig. C.9 of Appendix C.

When compared with the fixed base building, the base isolated and segmental structures with elasto-plastic and bilinear isolation devices have much smaller interstorey drifts for the Parkfield 1966 N65E earthquake as shown in Table 7.12. This is similar to results obtained for the buildings with additional damping having elasto-plastic and bilinear isolation systems as

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.52%	0.25%	0.22%	0.11%	0.11%
11	0.74%	0.33%	0.33%	0.14%	0.11%
10	0.96%	0.36%	0.38%	0.16%	0.11%
9	0.88%	0.41%	0.44%	0.14%	0.16%
8	0.96%	0.19%	0.22%	0.14%	0.19%
7	0.63%	0.30%	0.30%	0.11%	0.22%
6	0.63%	0.41%	0.44%	0.11%	0.11%
5	0.79%	0.47%	0.49%	0.11%	0.11%
4	1.18%	0.49%	0.49%	0.36%	0.36%
3	1.53%	0.47%	0.47%	0.41%	0.47%
2	1.75%	0.41%	0.44%	0.44%	0.44%
1	1.60%	0.26%	0.26%	0.32%	0.32%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.52%	0.36%	0.33%	0.14%	0.16%
11	0.74%	0.41%	0.33%	0.11%	0.14%
10	0.96%	0.38%	0.47%	0.11%	0.11%
9	0.88%	0.44%	0.52%	0.11%	0.11%
8	0.96%	0.41%	0.33%	0.22%	0.27%
7	0.63%	0.33%	0.33%	0.19%	0.22%
6	0.63%	0.44%	0.49%	0.19%	0.25%
5	0.79%	0.52%	0.60%	0.25%	0.27%
4	1.18%	0.66%	0.66%	0.55%	0.55%
3	1.53%	0.68%	0.71%	0.71%	0.74%
2	1.75%	0.71%	0.71%	0.82%	0.82%
1	1.60%	0.46%	0.46%	0.64%	0.64%

(b) Structures Mounted on Bilinear Isolation Systems

Table 7.10 Interstorey Drifts of Structures for the Taft 1952 N69W Earthquake

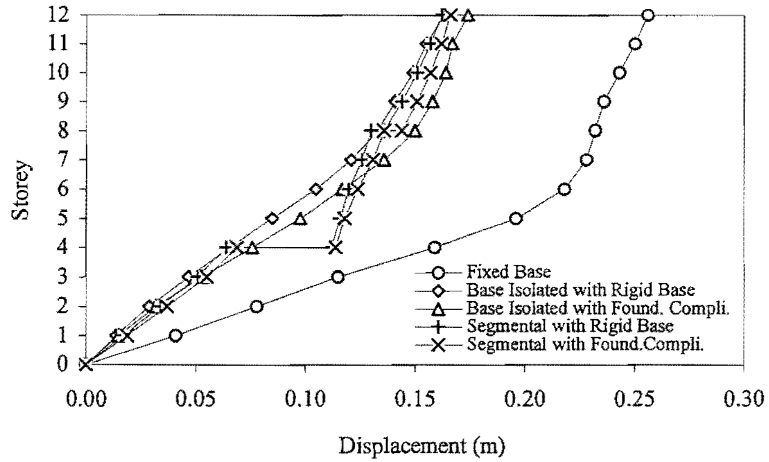
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.27%	0.14%	0.16%	0.14%	0.16%
11	0.27%	0.22%	0.22%	0.22%	0.22%
10	0.19%	0.27%	0.30%	0.19%	0.25%
9	0.30%	0.30%	0.33%	0.11%	0.11%
8	0.47%	0.27%	0.36%	0.19%	0.22%
7	0.63%	0.27%	0.38%	0.22%	0.27%
6	0.82%	0.38%	0.38%	0.11%	0.22%
5	1.04%	0.41%	0.41%	0.11%	0.11%
4	1.12%	0.41%	0.41%	0.30%	0.30%
3	1.18%	0.44%	0.41%	0.36%	0.36%
2	1.32%	0.36%	0.38%	0.36%	0.36%
1	1.14%	0.22%	0.22%	0.24%	0.26%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

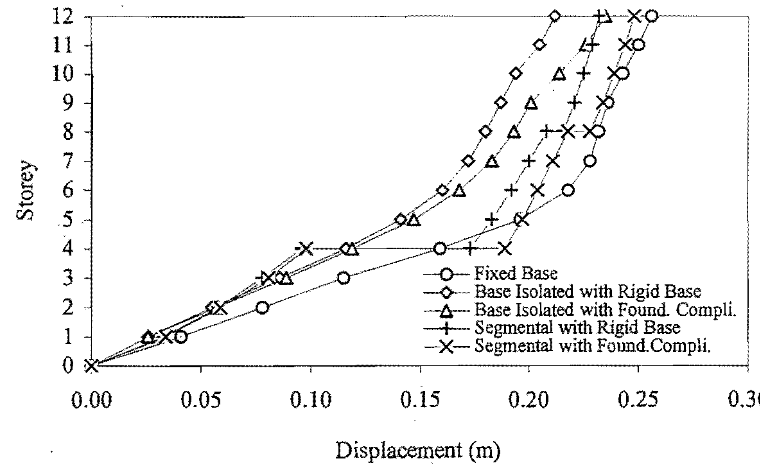
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.27%	0.14%	0.19%	0.08%	0.14%
11	0.27%	0.22%	0.25%	0.14%	0.16%
10	0.19%	0.30%	0.36%	0.08%	0.08%
9	0.30%	0.33%	0.36%	0.08%	0.08%
8	0.47%	0.36%	0.41%	0.19%	0.25%
7	0.63%	0.41%	0.44%	0.19%	0.30%
6	0.82%	0.44%	0.47%	0.25%	0.27%
5	1.04%	0.52%	0.49%	0.27%	0.30%
4	1.12%	0.55%	0.58%	0.52%	0.52%
3	1.18%	0.63%	0.63%	0.68%	0.68%
2	1.32%	0.60%	0.63%	0.71%	0.74%
1	1.14%	0.42%	0.42%	0.56%	0.56%

(b) Structures Mounted on Bilinear Isolation Systems

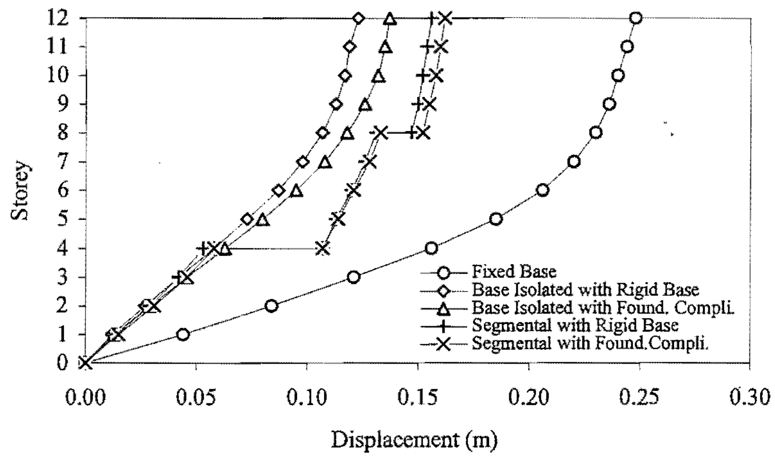
Table 7.11 Interstorey Drifts of the Structures with Additional Damping for the Taft 1952 N69W Earthquake



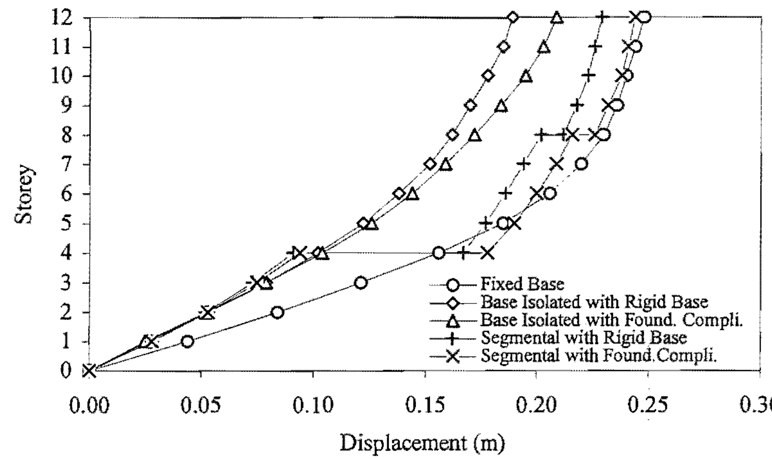
(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems



(c) Structures with Additional Damping Mounted on Elasto-Plastic Isolation Systems



(d) Structures with Additional Damping Mounted on Bilinear Isolation Systems

Fig. 7.5 Comparisons of Displacement with Storey of Different Structures for the Parkfield 1966 N65E Earthquake shown in Table 7.13. Similar results are also seen for the structures with and without additional damping based on NZS 3101:1982 as shown in Tables C.7 and C.8 of Appendix C respectively.

Fig.7.6 (a) and (b) show that the storey displacements for the base isolated buildings with elasto-plastic and bilinear isolators are much smaller than those for the fixed base buildings under the Mexico 1985 SCT S00E earthquake. This is similar to results obtained for the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolators as shown in Fig.7.6 (c) and (d). However, the segmental buildings using elasto-plastic isolation systems have slightly over 2.5% interstorey deflections between the lower two segments as given in Fig. 7.6 (a). With the inclusion of the extra dampers, the segmental buildings using elasto-plastic isolation systems have reduced interstorey deflection below 2.5% between the lower two segments for the Mexico 1985 SCT S00E earthquake as shown in Fig. 7.6 (c).

For the structures designed to NZS 3101:1982, the base isolated and segmental buildings with and without additional damping mounted on elasto-plastic and bilinear isolation systems have reduced interstorey deflections when compared with the fixed base building under the Mexico 1985 SCT S00E earthquake as shown in Fig. C.10 of Appendix C. There are greater interstorey deflections between the lower two segments of the segmental buildings mounted on elasto-plastic and bilinear isolation devices, but they do not exceed the 2.5% interstorey deflection limit specified by NZS 4203:1992.

From Table 7.14, the interstorey drifts of the base isolated and segmental buildings with elasto-plastic and bilinear isolation systems are much smaller than those for the fixed base building for the Mexico 1985 SCT S00E earthquake. Similar results are also obtained for the base isolated and segmental structures with additional damping using elasto-plastic and bilinear isolation systems as shown in Table 7.15. For the structures designed to NZS 3101:1982, the structures with and without additional damping using elasto-plastic and bilinear isolation systems have strongly reduced interstorey drifts when compared with the fixed base building for the Mexico 1985 SCT S00E earthquake as shown in Tables C.9 and C.10 of Appendix C.

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.16%	0.22%	0.19%	0.14%	0.11%
11	0.19%	0.16%	0.11%	0.16%	0.14%
10	0.19%	0.22%	0.22%	0.19%	0.16%
9	0.11%	0.22%	0.22%	0.22%	0.19%
8	0.11%	0.33%	0.38%	0.11%	0.14%
7	0.27%	0.44%	0.52%	0.16%	0.19%
6	0.60%	0.55%	0.52%	0.11%	0.16%
5	1.01%	0.52%	0.60%	0.11%	0.11%
4	0.93%	0.52%	0.63%	0.36%	0.38%
3	1.01%	0.44%	0.52%	0.47%	0.49%
2	1.01%	0.41%	0.44%	0.47%	0.49%
1	0.82%	0.28%	0.30%	0.34%	0.38%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.16%	0.19%	0.25%	0.08%	0.11%
11	0.19%	0.30%	0.33%	0.11%	0.14%
10	0.19%	0.19%	0.36%	0.11%	0.14%
9	0.11%	0.19%	0.22%	0.14%	0.16%
8	0.11%	0.33%	0.27%	0.22%	0.19%
7	0.27%	0.33%	0.41%	0.22%	0.19%
6	0.60%	0.52%	0.58%	0.25%	0.19%
5	1.01%	0.68%	0.77%	0.27%	0.22%
4	0.93%	0.82%	0.82%	0.49%	0.47%
3	1.01%	0.85%	0.88%	0.55%	0.60%
2	1.01%	0.79%	0.85%	0.68%	0.68%
1	0.82%	0.52%	0.52%	0.66%	0.68%

(b) Structures Mounted on Bilinear Isolation Systems

Table 7.12 Interstorey Drifts of Structures for the Parkfield 1966 N65E Earthquake

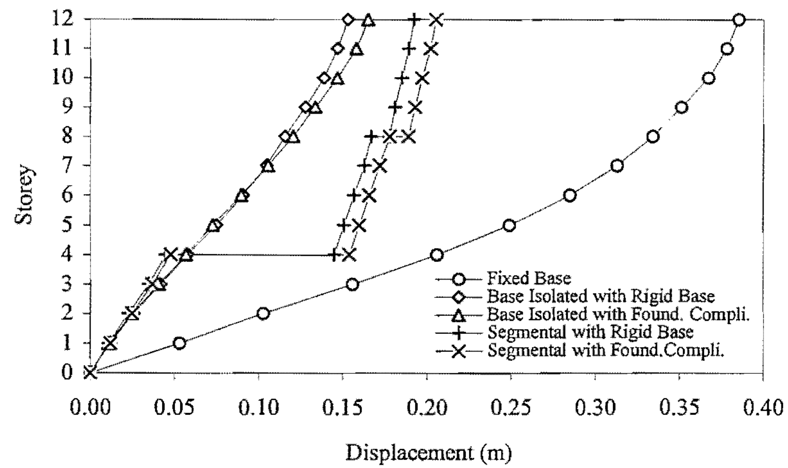
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.11%	0.11%	0.08%	0.08%	0.08%
11	0.11%	0.08%	0.08%	0.08%	0.08%
10	0.11%	0.11%	0.16%	0.08%	0.08%
9	0.16%	0.16%	0.22%	0.08%	0.08%
8	0.27%	0.25%	0.27%	0.14%	0.14%
7	0.38%	0.30%	0.36%	0.16%	0.19%
6	0.58%	0.38%	0.41%	0.19%	0.19%
5	0.79%	0.41%	0.47%	0.16%	0.19%
4	0.96%	0.44%	0.47%	0.30%	0.33%
3	1.01%	0.41%	0.47%	0.36%	0.41%
2	1.10%	0.41%	0.44%	0.41%	0.44%
1	0.88%	0.24%	0.26%	0.28%	0.30%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

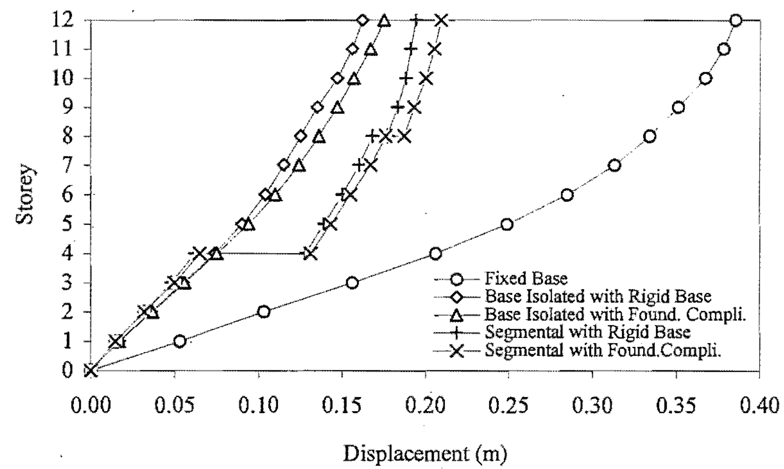
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.11%	0.11%	0.16%	0.08%	0.08%
11	0.11%	0.19%	0.22%	0.08%	0.08%
10	0.11%	0.22%	0.30%	0.14%	0.16%
9	0.16%	0.22%	0.33%	0.16%	0.16%
8	0.27%	0.27%	0.36%	0.22%	0.19%
7	0.38%	0.38%	0.41%	0.22%	0.25%
6	0.58%	0.44%	0.49%	0.25%	0.27%
5	0.79%	0.55%	0.60%	0.27%	0.33%
4	0.96%	0.63%	0.68%	0.49%	0.52%
3	1.01%	0.71%	0.74%	0.58%	0.60%
2	1.10%	0.74%	0.74%	0.66%	0.68%
1	0.88%	0.52%	0.50%	0.56%	0.56%

(b) Structures Mounted on Bilinear Isolation Systems

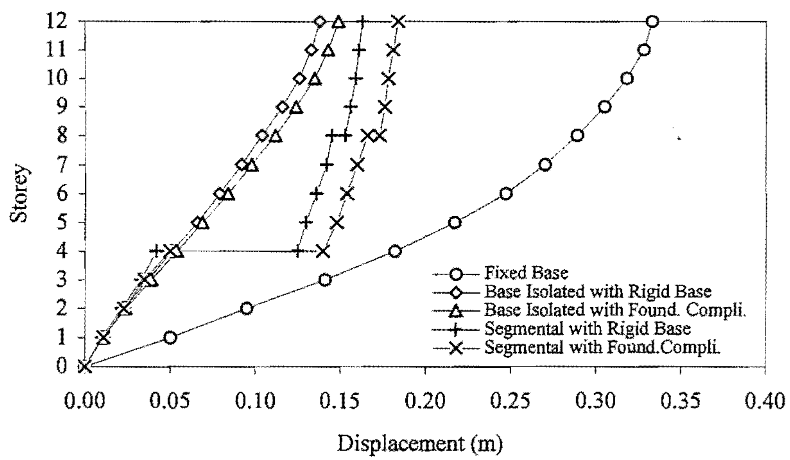
Table 7.13 Interstorey Drifts of Structures with Additional Damping for the Parkfield 1966 N65E Earthquake



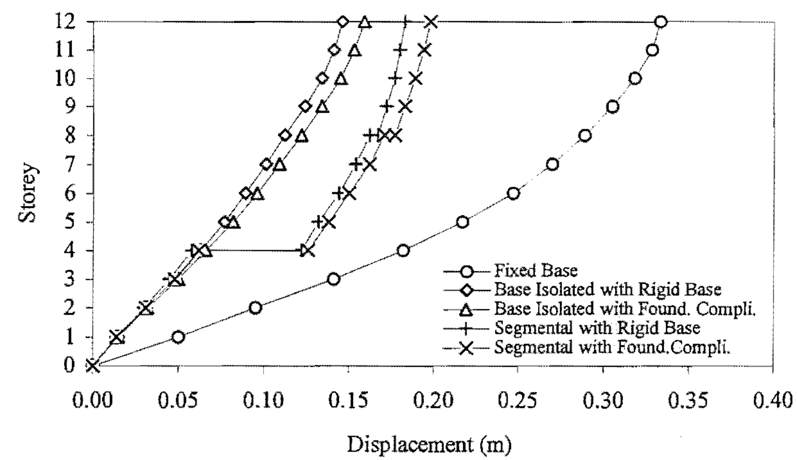
(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems



(c) Structures with Additional Damping Mounted on Elasto-Plastic Isolation Systems



(d) Structures with Additional Damping Mounted on Bilinear Isolation Systems

Fig. 7.6 Comparisons of Displacement with Storey of Different Structures for the Mexico 1985 SCT S00E Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.19%	0.16%	0.19%	0.08%	0.08%
11	0.30%	0.22%	0.30%	0.11%	0.14%
10	0.44%	0.30%	0.36%	0.11%	0.11%
9	0.47%	0.33%	0.36%	0.11%	0.11%
8	0.58%	0.30%	0.41%	0.11%	0.16%
7	0.77%	0.38%	0.44%	0.16%	0.16%
6	0.99%	0.44%	0.47%	0.16%	0.16%
5	1.18%	0.47%	0.44%	0.16%	0.16%
4	1.37%	0.44%	0.44%	0.27%	0.30%
3	1.45%	0.44%	0.44%	0.33%	0.33%
2	1.37%	0.38%	0.38%	0.33%	0.36%
1	1.06%	0.24%	0.22%	0.22%	0.24%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.19%	0.16%	0.22%	0.08%	0.11%
11	0.30%	0.25%	0.27%	0.08%	0.14%
10	0.44%	0.33%	0.27%	0.14%	0.19%
9	0.47%	0.27%	0.30%	0.16%	0.16%
8	0.58%	0.27%	0.33%	0.22%	0.25%
7	0.77%	0.30%	0.38%	0.27%	0.33%
6	0.99%	0.38%	0.44%	0.30%	0.33%
5	1.18%	0.44%	0.52%	0.30%	0.33%
4	1.37%	0.52%	0.52%	0.38%	0.45%
3	1.45%	0.52%	0.52%	0.44%	0.49%
2	1.37%	0.52%	0.55%	0.47%	0.47%
1	1.06%	0.34%	0.34%	0.30%	0.30%

(b) Structures Mounted on Bilinear Isolation Systems

Table 7.14 Interstorey Drifts of Structures for the Mexico 1985 SCT S00E Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.14%	0.16%	0.08%	0.08%
11	0.27%	0.19%	0.22%	0.08%	0.08%
10	0.36%	0.27%	0.30%	0.08%	0.08%
9	0.44%	0.33%	0.33%	0.08%	0.08%
8	0.52%	0.33%	0.38%	0.11%	0.16%
7	0.63%	0.36%	0.38%	0.16%	0.16%
6	0.82%	0.36%	0.41%	0.16%	0.16%
5	0.96%	0.41%	0.41%	0.14%	0.22%
4	1.12%	0.38%	0.41%	0.25%	0.41%
3	1.26%	0.38%	0.41%	0.30%	0.33%
2	1.23%	0.33%	0.36%	0.33%	0.33%
1	1.00%	0.22%	0.22%	0.20%	0.22%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.14%	0.16%	0.08%	0.11%
11	0.27%	0.19%	0.22%	0.08%	0.14%
10	0.36%	0.27%	0.30%	0.14%	0.16%
9	0.44%	0.33%	0.33%	0.14%	0.16%
8	0.52%	0.30%	0.36%	0.22%	0.25%
7	0.63%	0.33%	0.36%	0.27%	0.33%
6	0.82%	0.33%	0.38%	0.33%	0.33%
5	0.96%	0.38%	0.44%	0.27%	0.33%
4	1.12%	0.41%	0.44%	0.36%	0.38%
3	1.26%	0.47%	0.49%	0.41%	0.47%
2	1.23%	0.44%	0.47%	0.44%	0.47%
1	1.00%	0.30%	0.30%	0.28%	0.28%

(b) Structures Mounted on Bilinear Isolation Systems

Table 7.15 Interstorey Drifts of Structures with Additional Damping for the Mexico 1985 SCT S00E Earthquake

7.3.4 Base Shears

From the base shears obtained by the time history analyses shown in Table 7.16, the ratios of base shear to total weight of the fixed base buildings are respectively 6.89%, 7.48%, 7.16% and 5.95% under the El Centro 1940 N-S, Taft 1952 N69W, Parkfield 1966 N65E and Mexico 1985 SCT S00E earthquakes. Compared with Table 7.16, there is a slight difference (less than 10%) in the base shears for the fixed base buildings with additional damping for all four earthquakes as shown in Table 7.17. As observed from Tables 7.16 and 7.17, there is not a significant difference (less than 5%) in the base shears for the base isolated and segmental structures with and without additional damping using elasto-plastic and bilinear isolation systems on a rigid base and a compliant foundation for all four earthquakes. Similar results are seen for the structures designed to NZS 3101:1982 as shown in Tables C.11 and C.12.

As shown in Table 7.16, the base shears of the base isolated and segmental buildings with elasto-plastic and bilinear isolation systems are reduced by approximately 33%, 19%, 45% and 37% for the El Centro 1940 N-S, 38%, 12%, 39% and 23% for the Taft 1952 N69W, 30%, 7%, 31% and 8% for the Parkfield 1966 N65E, and 36%, 10%, 42% and 28% for the Mexico 1985 SCT S00E earthquakes when compared with the base shears of the fixed base buildings. Table C.11 of Appendix C shows similar results for the structures designed to NZS 3101:1982.

From Table 7.17, the base shears for the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolation devices are reduced by approximately 25%, 5%, 11% and 6% for the El Centro 1940 N-S, 16%, 15%, 16% and 18% for the Taft 1952 N69W, 14%, 2%, 11% and 14% for the Parkfield 1966 N65E, and 7%, 2%, 7% and 3% for the Mexico 1985 SCT S00E earthquakes when compared with same buildings without additional damping as shown in Table 7.16. Similar results are seen for the structures designed to NZS 3101:1982 when compared with Tables C.11 and C.12 of Appendix C.

From the results obtained above, the base isolated and segmental structures using elasto-plastic and bilinear isolation systems have significantly reduced base shears when compared with the fixed base buildings for all four earthquakes except in the case of the base isolated and segmental structures with bilinear isolation devices when subjected to the Parkfield 1966 N65E earthquake. Similar results are also seen for the base isolated and segmental structures with

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0689	0.0748	0.0716	0.0595
Base Isolated Building with Rigid base	0.0461	0.0473	0.0509	0.0401
Base Isolated Building with Foundation Compliance	0.0466	0.0454	0.0496	0.0382
Segmental Building with Rigid Base	0.0377	0.0464	0.0493	0.0348
Segmental Building with Foundation Compliance	0.0388	0.0457	0.0498	0.0357

(a) Buildings Mounted on Elasto-Plastic Isolation Systems

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0689	0.0748	0.0716	0.0595
Base Isolated Building with Rigid base	0.0572	0.0659	0.0678	0.0552
Base Isolated Building with Foundation Compliance	0.0559	0.0667	0.0668	0.0537
Segmental Building with Rigid Base	0.0434	0.0603	0.0672	0.0447
Segmental Building with Foundation Compliance	0.0438	0.0574	0.0661	0.0428

(b) Buildings Mounted on Bilinear Isolation Systems

Table 7.16 Normalized Base Shears of Structures for Different Earthquakes

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0626	0.0673	0.0693	0.0582
Base Isolated Building with Rigid base	0.0361	0.0397	0.0437	0.0373
Base Isolated Building with Foundation Compliance	0.0352	0.0389	0.0439	0.0362
Segmental Building with Rigid Base	0.0346	0.0392	0.0437	0.0331
Segmental Building with Foundation Compliance	0.0344	0.0397	0.0444	0.0332

(a) Buildings Mounted on Elasto-Plastic Isolation Systems

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0626	0.0673	0.0693	0.0582
Base Isolated Building with Rigid base	0.0552	0.0572	0.0667	0.0541
Base Isolated Building with Foundation Compliance	0.0529	0.0564	0.0659	0.0528
Segmental Building with Rigid Base	0.0433	0.0494	0.0583	0.0432
Segmental Building with Foundation Compliance	0.0413	0.0484	0.0566	0.0414

(b) Buildings Mounted on Bilinear Isolation Systems

Table 7.17 Normalized Base Shears of Structures with Additional Damping for Different Earthquakes additional damping. This phenomenon is caused by the combination effects of the fundamental period shift, the additional hysteretic damping and the shape of the acceleration spectra which have lower magnitudes for long period (greater than 2 seconds) structures under the El Centro 1940 N-S, Taft 1952 N69W and Parkfield 1966 N65E earthquakes. For longer period structures under the Mexico 1985 SCT S00E earthquake, the fundamental periods may be shifted beyond the region of the peak spectral accelerations into the descending part of the spectrum. Therefore, lower base shears can be obtained for the base isolated and segmental structures.

These results agree with the conclusions of Andriono (1990) [A2] that the base shear of the structure with a base isolation system is less than that of the fixed base building except for the Mexico 1985 SCT S00E earthquake. Andriono used a structure with a short fundamental period (less than 1.0 second) to investigate the seismic responses of his structure.

7.4 Curvature Ductility Demands of Beams and Columns

As discussed in Chapters 5 and 6, the member ductility demand is the most common damage parameter for showing failure or not of beam members. In order to quantify the different member ductility demands, the maximum curvature ductility demands of beam ends and column bases for the different structures which were designed according to NZS 3101:1995, subjected to the El Centro 1940 N-S, Taft 1952 N69W, Parkfield 1966 N65E and Mexico 1985 SCT S00E earthquakes will be investigated in this section. The 12-storey reinforced concrete moment-resistant frame models are used as discussed in Sections 3.4.3 and 3.6. Carr and Tabuchi (1993) [C2] assumed that an ultimate curvature ductility factor of 30 can be used to evaluate the member damage index developed by Park and Ang [P3,P4]. As indicated in Chapter 5, the maximum curvature ductility factors of 20 and 30 for columns and beams are also used in this section.

a. Structures without Additional Damping

It can be observed from Table 7.18 that the curvature ductility demands at the internal and external column bases are 2.35 and 2.47 for the fixed base building, less than 1.0 for the base isolated and segmental buildings with a rigid base and compliant foundation mounted on elasto-plastic and bilinear isolation systems when subjected to the El Centro 1940 N-S earthquake. A maximum curvature ductility demand of less than 1.0 in the tables means that the member remains elastic, i.e. no plastic hinge occurs.

For the elasto-plastic isolation devices shown in Table 7.18 (a), the maximum beam curvature ductility demands for the base isolated and segmental buildings with a rigid base and a compliant foundation are reduced by approximately 73%, 69%, 69% and 67% when compared with the fixed base buildings under the El Centro 1940 N-S earthquake. Table 7.18 (b) shows very similar results for the base isolated and segmental buildings mounted on bilinear isolation devices. Similar results are also seen for the structures designed to NZS 3101:1982 as shown in Table C.13 (a) and (b) of Appendix C

From Table 7.19, the curvature ductility demands for the internal and external column bases are 4.73 and 5.17 for the fixed base building, less than 1.0 for the base isolated and segmental structures with a rigid base and a compliant foundation mounted on elasto-plastic and bilinear isolation devices when subjected to the Taft 1952 N69W earthquake.

When compared with the fixed base building shown in Table 7.19 (a), the base isolated and segmental buildings with a rigid base and a compliant foundation using elasto-plastic isolation systems show 68%, 71%, 88% and 83% reduction of the maximum beam curvature ductility demands under the Taft 1952 N69W earthquake. From Table 7.19 (b), the maximum beam curvature ductility demands for the base isolated and segmental buildings with a rigid base and a compliant foundation using bilinear isolation devices are reduced by approximately 57%, 61%, 68% and 66% when compared with the fixed base building.

For the structures with elasto-plastic isolation systems based on NZS 3101:1982, the maximum beam curvature ductility demands for the base isolated and segmental buildings with a rigid base and a compliant foundation are respectively 55%, 53%, 72% and 65% smaller than those for the fixed base buildings when subjected to the Taft 1952 N69W earthquake as shown in Table C.14 (a) of Appendix C. Similar results are seen for the base isolated and segmental buildings using bilinear isolation systems as shown in Table C.14 (b) of Appendix C.

From Table 7.20, the curvature ductility demands at the internal and external column bases are 2.58 and 2.46 for the fixed base building and less than 1.0 for the base isolated and segmental

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	2.24	< 1.00	< 1.00	< 1.00	1.02
	11	3.88	1.36	1.56	2.05	2.32
	10	5.41	1.59	1.75	2.35	2.46
	9	5.28	2.05	2.38	2.32	2.52
	8	4.69	1.41	1.81	< 1.00	< 1.00
	7	5.65	1.16	1.56	< 1.00	1.21
	6	6.40	< 1.00	1.06	< 1.00	< 1.00
	5	6.64	< 1.00	< 1.00	< 1.00	< 1.00
	4	6.45	< 1.00	< 1.00	< 1.00	< 1.00
	3	7.38	< 1.00	< 1.00	< 1.00	1.01
	2	7.59	< 1.00	< 1.00	< 1.00	1.09
	1	7.43	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L. Ext.*	2.47	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.35	< 1.00	< 1.00	< 1.00	< 1.00
	R. Ext.*	2.42	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	2.24	< 1.00	1.02	< 1.00	1.27
	11	3.88	1.87	1.92	2.49	2.59
	10	5.41	1.98	2.03	2.71	2.83
	9	5.28	2.12	2.18	2.65	2.87
	8	4.69	1.34	1.72	< 1.00	< 1.00
	7	5.65	1.41	1.73	< 1.00	1.22
	6	6.40	1.27	1.53	< 1.00	< 1.00
	5	6.64	1.50	1.50	< 1.00	< 1.00
	4	6.45	1.17	1.46	< 1.00	< 1.00
	3	7.38	1.49	1.87	1.34	1.67
	2	7.59	1.68	2.04	1.35	1.52
	1	7.43	1.51	1.69	1.26	1.38
Column Bases	L. Ext.*	2.47	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.35	< 1.00	< 1.00	< 1.00	< 1.00
	R. Ext.*	2.42	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.18 Maximum Curvature Ductility Demands of Structures for the El Centro 1940 N-S Earthquake

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	3.67	1.14	1.41	< 1.00	< 1.00
	11	5.76	2.60	2.70	1.10	1.58
	10	6.41	2.99	2.74	1.20	1.71
	9	7.32	3.59	3.18	1.36	1.91
	8	7.87	2.56	2.19	< 1.00	< 1.00
	7	8.17	2.04	2.36	< 1.00	1.15
	6	8.21	1.62	1.88	< 1.00	< 1.00
	5	8.23	1.31	1.53	< 1.00	< 1.00
	4	8.73	1.43	1.44	< 1.00	< 1.00
	3	8.82	1.08	1.23	< 1.00	1.42
	2	10.12	< 1.00	1.28	1.26	1.64
	1	11.13	< 1.00	1.01	1.19	1.53
Column Bases	L.Ext.*	5.09	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	4.73	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	5.17	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	3.67	1.33	1.52	< 1.00	< 1.00
	11	5.76	2.40	2.56	1.38	1.88
	10	6.41	3.15	3.28	1.34	2.02
	9	7.32	4.75	4.38	1.90	2.48
	8	7.87	3.76	3.68	< 1.00	< 1.00
	7	8.17	3.04	3.69	1.48	2.13
	6	8.21	2.49	2.89	1.02	1.49
	5	8.23	2.85	3.04	1.03	1.51
	4	8.73	2.99	3.16	1.21	1.39
	3	8.82	3.12	3.17	2.49	2.76
	2	10.12	2.91	3.03	3.45	3.73
	1	11.13	2.51	2.65	3.58	3.76
Column Bases	L.Ext.*	5.09	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	4.73	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	5.17	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.19 Maximum Curvature Ductility Demands of Structures for the Taft 1952 N69W Earthquake structures with a rigid base and compliant foundation using elasto-plastic and bilinear isolation devices under the Parkfield 1966 N65E earthquake.

As observed from Table 7.20 (a), the maximum beam curvature ductility demands for the base isolated and segmental buildings with a rigid base and compliant foundation mounted on elasto-plastic isolation systems show respectively 59%, 58%, 85% and 80% reduction when compared with the fixed base buildings when subjected to the Parkfield 1966 N65E earthquake. From Table 7.20 (b), the base isolated and segmental buildings with a rigid base and compliant foundation using bilinear isolation devices show respectively 49%, 47%, 64% and 61% reduction of the maximum beam curvature ductility demands when compared with the fixed base building.

For the structures designed to NZS 3101:1982 as shown in Table C.15 (a) of Appendix C, the maximum beam curvature ductility demands for the base isolated and segmental buildings mounted on elasto-plastic isolation systems are reduced by approximately 61%, 62%, 67% and 62% when compared the fixed base buildings under the Parkfield 1966 N65E earthquake. Table C.15 (b) of Appendix C shows very similar results for the base isolated and segmental structures with bilinear isolation devices.

From Table 7.21, the curvature ductility demands at the internal and external column bases are 2.68 and 2.81 for the fixed base building and less than 1.0 for the base isolated and segmental structures with a rigid base and compliant foundation using elasto-plastic and bilinear isolation devices when subjected to the Mexico 1985 SCT S00E earthquake.

When compared with fixed base building as shown in Table 7.21 (a), the base isolated and segmental buildings with a rigid base and compliant foundation using elasto-plastic isolation systems show respectively 80%, 76%, 89% and 88% reduction of the maximum beam curvature ductility demands when subjected to the Mexico 1985 SCT S00E earthquake. Table 7.21 (b) shows very similar results for the base isolated and segmental buildings mounted on bilinear isolation devices. Similar results are also observed for the structures designed to NZS 3101:1982 as shown in Table C.16 (a) and (b) of Appendix C.

From the results obtained above, the maximum beam curvature ductility demands for the base isolated and segmental structures with a rigid base and a compliant foundation show

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	1.13	< 1.00	1.19	< 1.00	< 1.00
	11	2.69	2.25	2.64	< 1.00	1.09
	10	3.74	2.65	2.67	< 1.00	1.10
	9	4.11	2.30	2.34	< 1.00	1.32
	8	3.29	1.03	1.07	< 1.00	< 1.00
	7	2.87	1.20	1.54	< 1.00	< 1.00
	6	3.33	1.12	1.56	< 1.00	< 1.00
	5	5.59	1.41	1.97	< 1.00	< 1.00
	4	6.49	1.51	2.02	< 1.00	< 1.00
	3	6.48	1.24	1.78	< 1.00	< 1.00
	2	5.58	< 1.00	1.42	< 1.00	1.20
	1	5.87	< 1.00	< 1.00	< 1.00	1.13
Column Bases	L.Ext.*	2.44	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.58	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.46	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	1.13	1.01	1.29	< 1.00	< 1.00
	11	2.69	2.60	2.72	< 1.00	1.15
	10	3.74	2.84	2.81	< 1.00	1.13
	9	4.11	2.56	2.45	1.14	1.33
	8	3.29	1.11	1.65	< 1.00	< 1.00
	7	2.87	1.15	1.75	< 1.00	1.28
	6	3.33	1.19	1.61	< 1.00	< 1.00
	5	5.59	1.87	2.31	< 1.00	1.14
	4	6.49	2.52	2.92	< 1.00	< 1.00
	3	6.48	3.06	3.42	1.66	1.82
	2	5.58	3.29	3.47	1.89	1.99
	1	5.87	2.41	2.48	2.35	2.50
Column Bases	L.Ext.*	2.44	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.58	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.46	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.20 Maximum Curvature Ductility Demands of Structures for the Parkfield 1966 N65E Earthquake

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	2.24	1.23	1.59	< 1.00	< 1.00
	10	3.50	1.43	1.73	< 1.00	< 1.00
	9	4.52	1.87	2.18	< 1.00	1.07
	8	4.96	1.30	1.56	< 1.00	< 1.00
	7	5.68	1.48	1.74	< 1.00	< 1.00
	6	6.11	1.05	1.24	< 1.00	< 1.00
	5	7.19	1.09	1.21	< 1.00	< 1.00
	4	8.27	1.13	1.20	< 1.00	< 1.00
	3	9.26	1.07	1.13	< 1.00	< 1.00
	2	8.85	< 1.00	1.01	< 1.00	< 1.00
	1	8.09	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	2.77	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.68	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.81	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	2.24	1.43	1.80	< 1.00	< 1.00
	10	3.50	1.63	1.92	< 1.00	< 1.00
	9	4.52	1.95	2.31	< 1.00	< 1.00
	8	4.96	1.24	1.54	< 1.00	< 1.00
	7	5.68	1.44	1.67	< 1.00	1.07
	6	6.11	1.05	1.24	< 1.00	< 1.00
	5	7.19	1.15	1.33	< 1.00	< 1.00
	4	8.27	1.33	1.45	< 1.00	< 1.00
	3	9.26	1.47	1.54	1.12	1.47
	2	8.85	1.46	1.61	1.32	1.58
	1	8.09	1.27	1.35	1.15	1.32
Column Bases	L.Ext.*	2.77	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.68	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.81	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.21 Maximum Curvature Ductility Demands of Structures for the Mexico 1985 SCT S00E Earthquake insignificant difference under all four earthquakes. This is true for both the elasto-plastic and bilinear isolation systems. For the structures designed to NZS 3101:1995 and NZS 3101:1982, the base isolated and segmental building have significantly reduced maximum beam curvature ductility demands when compared with the fixed base building for all four earthquakes.

b. Structures with Additional Damping

As may be seen from Tables 7.22, 7.23, 7.24 and 7.25, the fixed base buildings with additional damping have reduced curvature ductility demands at the column bases when compared with the buildings without additional damping under the El Centro 1940 N-S, Taft 1952 N69W, Parkfield 1966 N65E and Mexico 1985 SCT S00E earthquakes as shown in Tables 7.18, 7.19, 7.20 and 7.21 respectively. Similar effects can be seen for the fixed base buildings with additional damping when designed to NZS 3101:1982 as shown in Tables C.17, C.18, C.19 and C.20 of Appendix C when compared with the buildings without additional damping for the four earthquakes as shown in Tables C.13, C.14, C.15 and C.16 of Appendix C respectively.

From Tables 7.22, 7.23, 7.24 and 7.25, it can be seen that the maximum beam curvature ductility demands for the fixed base buildings with additional damping are reduced by approximately 6%, 19%, 7% and 11% when compared with the same buildings without additional damping under the four earthquakes as shown in Tables 7.18, 7.19, 7.20 and 7.21.

For the structures designed to NZS 3101:1982 shown in Tables C.17, C.18, C.19 and C.20 of Appendix C, the fixed base buildings with additional damping show 19%, 21%, 28% and 23% reduction of maximum beam curvature ductility demands when compared with the same buildings without additional damping for the four earthquakes as shown in Tables C.13, C.14, C.15 and C.16 of Appendix C respectively.

From the results obtained above, there are greater reductions in the maximum beam curvature ductility demands for the fixed base building designed to NZS 3101:1982 when compared with the same building designed to NZS 3101:1995 for all four earthquakes.

As shown in Tables 7.22, 7.23, 7.24 and 7.25, the maximum beam curvature ductility demands for the base isolated and segmental buildings with additional damping mounted on

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.39	< 1.00	< 1.00	< 1.00	1.09
	10	2.30	< 1.00	< 1.00	< 1.00	1.08
	9	3.09	1.07	1.38	< 1.00	1.30
	8	2.92	< 1.00	< 1.00	< 1.00	< 1.00
	7	3.95	< 1.00	1.03	< 1.00	< 1.00
	6	4.99	< 1.00	< 1.00	< 1.00	< 1.00
	5	5.54	< 1.00	< 1.00	< 1.00	< 1.00
	4	6.33	< 1.00	< 1.00	< 1.00	< 1.00
	3	6.92	< 1.00	< 1.00	< 1.00	< 1.00
	2	7.12	< 1.00	< 1.00	< 1.00	< 1.00
	1	7.01	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	2.33	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.27	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.35	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.39	< 1.00	< 1.00	< 1.00	1.27
	10	2.30	< 1.00	1.07	< 1.00	1.28
	9	3.09	1.05	1.45	1.10	1.48
	8	2.92	< 1.00	< 1.00	< 1.00	< 1.00
	7	3.95	< 1.00	1.26	< 1.00	< 1.00
	6	4.99	< 1.00	< 1.00	< 1.00	< 1.00
	5	5.54	< 1.00	1.08	< 1.00	< 1.00
	4	6.33	< 1.00	1.11	< 1.00	< 1.00
	3	6.92	1.03	1.37	1.03	1.28
	2	7.12	1.25	1.53	1.28	1.39
	1	7.01	1.15	1.28	1.08	1.19
Column Bases	L.Ext.*	2.33	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.27	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.35	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.22 Maximum Curvature Ductility Demands of Structures with Additional Damping for the El Centro 1940 N-S Earthquake

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	1.05	< 1.00	< 1.00	< 1.00	< 1.00
	11	2.68	1.01	1.15	< 1.00	1.14
	10	4.22	1.38	1.36	< 1.00	1.13
	9	5.35	1.90	1.81	< 1.00	1.35
	8	4.95	1.11	1.10	< 1.00	< 1.00
	7	5.14	1.06	1.29	< 1.00	< 1.00
	6	6.36	< 1.00	< 1.00	< 1.00	< 1.00
	5	7.03	< 1.00	1.03	< 1.00	< 1.00
	4	7.97	< 1.00	1.12	< 1.00	< 1.00
	3	8.71	< 1.00	1.08	< 1.00	< 1.00
	2	8.61	< 1.00	< 1.00	< 1.00	1.08
	1	9.07	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	3.06	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	3.07	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	3.12	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	1.05	< 1.00	< 1.00	< 1.00	< 1.00
	11	2.68	1.27	1.79	< 1.00	1.22
	10	4.22	1.86	2.19	< 1.00	1.23
	9	5.35	2.58	2.91	1.08	1.60
	8	4.95	1.66	2.07	< 1.00	< 1.00
	7	5.14	1.57	2.18	1.12	1.76
	6	6.36	1.48	1.90	< 1.00	1.32
	5	7.03	1.94	2.23	1.02	1.44
	4	7.97	2.39	2.56	< 1.00	< 1.00
	3	8.71	2.60	2.68	2.27	2.55
	2	8.61	2.52	2.58	2.97	3.18
	1	9.07	2.09	2.27	3.01	3.10
Column Bases	L.Ext.*	3.06	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	3.07	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	3.12	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.23 Maximum Curvature Ductility Demands of Structures with Additional Damping for the Taft 1952 N69W Earthquake

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.27	< 1.00	< 1.00	< 1.00	< 1.00
	10	2.05	< 1.00	1.05	< 1.00	< 1.00
	9	2.54	1.11	1.24	< 1.00	< 1.00
	8	2.09	< 1.00	< 1.00	< 1.00	< 1.00
	7	2.43	< 1.00	< 1.00	< 1.00	< 1.00
	6	3.45	< 1.00	< 1.00	< 1.00	< 1.00
	5	5.22	< 1.00	< 1.00	< 1.00	< 1.00
	4	5.63	< 1.00	< 1.00	< 1.00	< 1.00
	3	6.04	< 1.00	< 1.00	< 1.00	< 1.00
	2	6.02	< 1.00	< 1.00	< 1.00	< 1.00
	1	6.05	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	2.31	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.39	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.33	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.27	< 1.00	< 1.00	< 1.00	< 1.00
	10	2.05	< 1.00	< 1.00	< 1.00	< 1.00
	9	2.54	1.03	1.45	< 1.00	< 1.00
	8	2.09	< 1.00	1.18	< 1.00	< 1.00
	7	2.43	< 1.00	1.45	< 1.00	1.28
	6	3.45	< 1.00	1.11	< 1.00	< 1.00
	5	5.22	1.05	1.43	< 1.00	1.21
	4	5.63	1.52	1.86	< 1.00	< 1.00
	3	6.04	2.10	2.28	1.46	1.75
	2	6.02	2.37	2.49	1.88	2.06
	1	6.05	2.12	2.07	2.06	2.17
Column Bases	L.Ext.*	2.31	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.39	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.33	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.24 Maximum Curvature Ductility Demands of Structures with Additional Damping for the Parkfield 1966 N65E Earthquake

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.45	< 1.00	< 1.00	< 1.00	< 1.00
	10	2.51	< 1.00	1.17	< 1.00	< 1.00
	9	3.56	1.34	1.66	< 1.00	< 1.00
	8	3.75	< 1.00	1.11	< 1.00	< 1.00
	7	4.61	1.03	1.28	< 1.00	< 1.00
	6	5.47	< 1.00	< 1.00	< 1.00	< 1.00
	5	6.02	< 1.00	< 1.00	< 1.00	< 1.00
	4	7.14	< 1.00	1.04	< 1.00	< 1.00
	3	7.81	< 1.00	< 1.00	< 1.00	< 1.00
	2	8.22	< 1.00	< 1.00	< 1.00	< 1.00
	1	7.68	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	2.61	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.68	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.64	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	11	1.45	< 1.00	< 1.00	< 1.00	< 1.00
	10	2.51	< 1.00	1.13	< 1.00	< 1.00
	9	3.56	1.34	1.62	< 1.00	< 1.00
	8	3.75	< 1.00	1.08	< 1.00	< 1.00
	7	4.61	1.04	1.29	< 1.00	< 1.00
	6	5.47	< 1.00	< 1.00	< 1.00	< 1.00
	5	6.02	< 1.00	1.09	< 1.00	< 1.00
	4	7.14	< 1.00	1.28	< 1.00	< 1.00
	3	7.81	1.11	1.43	< 1.00	1.38
	2	8.22	1.22	1.51	1.11	1.45
	1	7.68	1.06	1.21	< 1.00	1.18
Column Bases	L.Ext.*	2.61	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	2.68	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.64	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table 7.25 Maximum Curvature Ductility Demands of Structures with Additional Damping for the Mexico 1985 SCT S00E Earthquake elasto-plastic and bilinear isolation systems have significant reductions when compared with the same buildings without additional damping for all four earthquakes as given in Tables 7.18, 7.19, 7.20 and 7.21 except in the case of the base isolated and segmental buildings with bilinear isolation systems under the Taft 1952 N69W and Parkfield 1966 N65E earthquakes.

For the structures designed to NZS 3101:1982 shown in Tables C.17, C.18, C.19 and C.20 of Appendix C, the base isolated and segmental buildings with additional damping have significantly reduced maximum beam curvature ductility demands when compared with the same buildings without additional damping for all four different earthquakes as shown in Tables C.13, C.14, C.15 and C.16 of Appendix C except in the case of the base isolated and segmental buildings with bilinear isolation systems for the Parkfield 1966 N65E earthquake.

7.5 Summary and Conclusion

A series of the time history analyses have been carried out to investigate in detail the seismic responses of the fixed base, base isolated and segmental structures with and without additional damping mounted on elasto-plastic and bilinear isolation systems when subjected to the El Centro 1940 N-S, Taft 1952 N69W, Parkfield 1966 N65E and Mexico 1985 SCT S00E earthquakes. The first three earthquakes have peak spectral accelerations in the short period region and the last ground motion has its peak spectral accelerations at longer periods.

In Section 7.2, a method, which is based on different weights in the fundamental, second and third modal periods of the structures, for calculating the scale factors for the four chosen earthquakes was developed. In the time history analyses, the earthquake records were scaled according to their 5% damped spectra to match the design spectrum in Section 4.6.2.9 (b) (ii) of NZS 4203:1992 for the intermediate soil sites. The scale factors were obtained so that the base shears of the structures do not differ significantly under the four chosen earthquake records.

From the results mentioned in Section 7.3, the top floor displacements relative to their base floor displacements for the base isolated and segmental buildings are much smaller than the top displacements for the fixed base buildings for all four earthquakes. This is true for both the elasto-plastic and bilinear isolation systems. It can also be seen that the installation of extra damping devices in a structure will significantly reduced top floor displacements when compared with the structures without these additional damping devices. This is true for the structures designed to NZS 3101:1995 and NZS 3101:1982. These results confirm that the increased additional damping due to the hysteretic behaviour of the isolation devices reduces the lateral displacements for the base isolated and segmental structures.

As shown in Section 7.3, the storey displacements and interstorey drifts for the fixed base, base isolated and segmental structures with additional damping are much smaller than those without additional damping under the four earthquakes. For the structures with and without additional damping, the segmental buildings have smaller interstorey drifts than the base isolated buildings.

In Section 7.3, the structures with elasto-plastic and bilinear isolation systems when designed to NZS 3101:1995 indicate that the segmental buildings have smaller base shears than the base isolated buildings, and they give much smaller base shears than the fixed base building for all four earthquakes. Similar results are seen for the structures designed to NZS 3101:1982. There are insignificant differences in the base shears for the structures with additional damping when compared with the same buildings without additional damping.

It can be seen from Section 7.3 that there are significant reductions of base shears and interstorey drifts for the base isolated and segmental structures under the four chosen ground motions. This is because of the combination effects of the fundamental period shift, the additional hysteretic damping and the shape of the acceleration spectra which have lower magnitudes for long period structures under the El Centro 1940 N-S, Taft 1952 N69W and Parkfield 1966 N65E earthquakes. For longer period structures under the Mexico 1985 SCT S00E ground motion, the fundamental periods may be shifted beyond the region of the peak spectral accelerations into the descending part of the spectrum.

The isolator displacements for the base isolated and segmental buildings with elasto-plastic and bilinear isolation systems for the four chosen earthquakes are shown in Table 7.26. From the results obtained above, the isolator displacements of the base floor for the isolated buildings are greater than those for the segmental buildings. Each isolator displacement in the segmental building is smaller than the base isolator displacement in the base isolated building. This is true

Earthquake Records	Displacement of Isolation System (m)			
	Base Isolated Building	Segmental Building		
	Base Floor	Base Floor	Middle Segment	Top Segment
El Centro 1940 N-S	0.076	0.070	0.072	0.060
Taft 1952 N69W	0.326	0.237	0.246	0.053
Parkfield 1966 N65E	0.192	0.144	0.152	0.034
Mexico 1985 SCT S00E	0.120	0.066	0.106	0.074

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Earthquake Records	Displacement of Isolation System (m)			
	Base Isolated Building	Segmental Building		
	Base Floor	Base Floor	Middle Segment	Top Segment
El Centro 1940 N-S	0.074	0.038	0.049	0.038
Taft 1952 N69W	0.099	0.074	0.089	0.058
Parkfield 1966 N65E	0.106	0.079	0.092	0.046
Mexico 1985 SCT S00E	0.080	0.030	0.070	0.017

(b) Structures Mounted on Bilinear Isolation Systems

Table 7.26 Displacements of Isolation Systems for Structures Designed to NZS 3101:1995 for the Four Earthquake Records for the elasto-plastic and bilinear models of the isolation systems. Similar results are seen for the structures designed to NZS 3101:1982 as shown in Table C.21.

As can be seen from Table 7.26 under the first three earthquakes, which have peak spectral accelerations in the short period region, the maximum isolator displacements are respectively 0.326 and 0.246 metres for the base isolated and segmental buildings with elasto-plastic isolation systems under the Taft 1952 N69W earthquake. For the Mexico 1985 SCT S00E earthquake, which has its peak spectral accelerations at longer periods, the maximum isolator displacements are respectively 0.120 and 0.106 metres for the base isolated and segmental buildings with elasto-plastic isolation systems. Smaller isolator displacements are obtained for the structures designed to NZS 3101:1982 under the four earthquakes as shown Table C.21 of Appendix C.

In Section 7.4, the structures designed to NZS 3101:1995 indicate that the maximum beam curvature ductility demands for the base isolated and segmental buildings using elasto-plastic isolation systems are reduced by approximately 70% and 70% for the El Centro 1940 N-S, 70% and 85% for the Taft 1952 N69W, 60% and 85% for the Parkfield 1966 N65E, and 80% and 90% for the Mexico 1985 SCT S00E earthquakes when compared with the fixed base building. This is similar to results observed for the base isolated and segmental buildings with bilinear isolation devices. Similar results are also obtained for the structures designed to NZS 3101:1982.

As shown in Section 7.4, the base isolated and segmental buildings with additional damping using elasto-plastic and bilinear isolation systems have significantly reduced maximum beam curvature ductility demands when compared with the same buildings without additional damping under the four earthquakes.

CHAPTER 8

CONSIDERATIONS FOR THE ANALYSIS AND DESIGN OF BASE ISOLATED AND SEGMENTAL STRUCTURES

8.1 Introduction

It is emphasized that the analysis and design considerations are regarded as a design guide only, namely as a means of assisting the designer to understand the features of the long period structures such as the base isolated and segmental buildings and properties of the isolation devices at the initial design stage, which may then be used for the time history analyses.

8.2 Proposed Design Procedure

The primary purpose in the design of base isolated and segmental structures is that no damage or only minor damage occurs in the superstructure, and that inelastic deformations are concentrated in the isolation system during the earthquake attacks. The proposed design procedures for the base isolated and segmental buildings are shown as follows:

a. Base Isolated Building

Step 1: Determine the fundamental period of the un-isolated building.

For a preliminary design stage, the period of the un-isolated building should be established from properly substantiated data. The approximate formulae as recommended by some codes [C10,D4,S8,T1] could be used to estimate the fundamental period of this fixed base superstructure.

Step 2: Calculate the base shears induced by an earthquake and by wind loading.

The base shear force in the structure induced by an earthquake using the equivalent static method is compared with those forces induced by wind loading according to NZS 4203:1992 or similar codes. From the results shown in Table 4.1, the earthquake-induced base shear force must be greater than the wind-induced base shear force so that base isolation devices can be used as a feature of the seismic design of the structures.

Step 3: Determine the yield strength of the isolation system.

The yield strength of the isolator is required to exceed the level of the wind loading used in design by a margin to prevent yield under wind-storm conditions. At the same time, it is necessary to consider the limitations of rocking modes induced by the isolators. The yield strength of the isolator to the weight of the structure, F_y /W, may be used as described in Section 4.5 and Table 4.1.

Step 4: Design the base isolated building.

A design approach of the base isolated model is briefly stated below.

1. Select the building configuration including the above isolator, foundation and structural materials.
2. Determine the cross-sections of beams and columns of the superstructure and foundation.
3. Use the loading code to calculate the live load contributions to the seismic mass for each floor of the structural model.

Step 5: Determine the initial stiffness, post-yield stiffness and yield strength of the isolation system.

The initial stiffness of the isolator, k_o, may be taken as ten times the total weight, W of the structure per metre (10.0 W/m) as found in Section 4.5. The post-yield stiffness of the isolator, αk_o, may be used as 0.4W/m, i.e. $\alpha = 0.04$ for the bilinear model as found in Section 5.2. The yield strength, F_y, is given in Step 3.

Step 6: Perform the inelastic time history analyses.

Step 7: Check the displacements of the isolators.

If the isolator displacements using the time history analyses are greater than the displacement limits of the isolator, Step 5 should be repeated.

Step 8: Design the members of the structure.

b. Segmental Building

Steps 1 to 3 are the same as the design for the base isolated building.

Step 4: Design the segmental building.

After the base isolated model has been obtained, the segmental building model may be created as below.

1. Segment the floors of the superstructure for the base isolated model to the separate parts wanted, to form the segmental building model.
2. Design links between the two segments to prevent the occurrence of rocking modes of the structure and to transmit the gravity loads between the two segments.

Three possible means may be used to locate the isolators between two separated segments of the structure. If the isolators are located in a separate level between the segments the lateral displacement will be limited by the displacement limits of the devices as shown in Fig. 8.1 (a). The masses of floors n and n+1 between the two adjacent segments are considered as the floor masses at each level respectively. This level could also be used as a service level with a normal storey height instead of just an isolator level only with a crawl space between floors.

If the devices are located within a storey of the structure as shown in Fig. 8.1 (b), the 2.5% interstorey deflection limit (2.5% storey height) specified in NZS 4203:1992 could be used to limit the available device displacement. This may

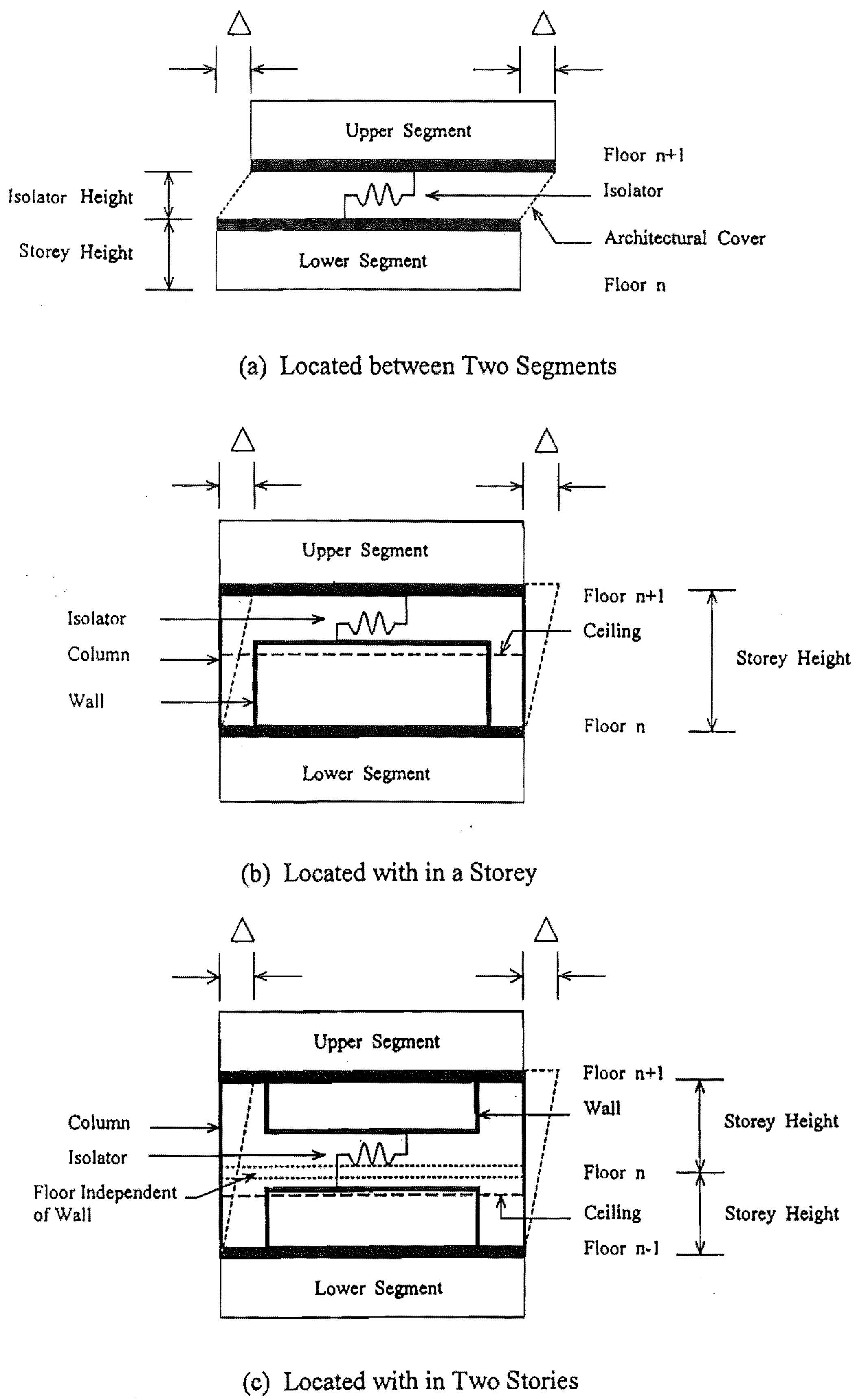


Fig 8.1 Different Isolation Installation Options for Segmental Structures

be a good starting point for the interstorey deflection limit in the design of segmental buildings. The masses of floors n and n+1 between the two adjacent segments are considered as the floor masses respectively but will include the mass of the isolators and the supporting structures. The deflection limit depends on the limits posed by the actual structural layout in order to limit damage to structural and non-structural components such as curtain walling, ceilings, partitions and building services.

If the devices are located within two stories of the structure as shown in Fig. 8.1 (c), the 2.5% interstorey deflection limit over the two stories (2.5% of two times storey height) will limit the available device displacement. The masses of floors n-1 and n+1 between the two adjacent segments are considered as carrying the isolators and their supporting structures whereas the mass of floor n is carried by the column members between floors n-1 and n+1. These columns could, for instance, be pinned at floors n-1 and n+1. The deflection limit depends more on the limits posed by the actual structural layout in order to limit damage to structural and non-structural components such as curtain walling, ceilings, partitions and building services.

In cases (b) and (c) above, the interconnection of services through the segment junctions should not be a problem any more than they are in a conventionally designed fixed base or base isolated structure as these buildings are controlled by the code interstorey drift limits. In case (a), although the segmental structures may be undamaged, the services (especially elevators), may require the dissipators being jacked back to near their original positions for the services to be fully operational. Other services should have similar connection problems to these provided in base isolated building and elevators could be supported over stories above and below the isolator as suggested in case (c) above. It was noted in Section 7.3.3 of Chapter 7 that the maximum displacements between the segments were 0.072, 0.246, 0.152 and 0.106 metres for the four different excitations whereas the code 2.5 % interstorey drift limit for a storey height of 3.65 metres is 0.091 metre.

Step 5: Determine the initial stiffness, post-yield stiffness and yield strength of the isolation system.

The post-yield stiffness of the isolation system, αk_o, may be taken as 0.5 W/m, i.e. $\alpha = 0.05$ for the bilinear model as found in Section 5.2. If the segmental building is divided into three segments as shown in Fig. 3.13 of Chapter 3, the following method may be used to determine the initial stiffness and strength of the isolation system according to the weight of the segments. For the isolation systems installed at the base floor, the total weight (3 segments) of the structure was used. For the isolators located at the base of the middle segment, the total weight of the top and middle segments was used. For the isolation systems placed at the base of the top segment, the combined weight of the top and 80% of the middle segment was used. The initial stiffnesses, k_o and yield strengths, F_y of the isolation systems for the base isolated and segmental buildings designed to NZS 3101:1995 and NZS 3101:1982 are summarised in Table 8.1.

Steps 6 to 8 are the same as the design for the base isolated building.

8.3 Example

a. Procedure for the Design of the Base Isolated Building

Step 1: For the building designed to NZS 3101:1982 and NZS 4203:1992, the fundamental period, $T_{(UI)}$, of 2.012 seconds and the structural ductility, μ, of 4.0 of the un-isolated frame building are based on the cracked section properties listed in Appendix D.

Step 2: From the ratios of base shear to weight of the structure shown in Table 4.1, the earthquake-induced base shear is 5.1% at the structural ductility of 4 which is greater than the wind-induced base shear of 2.8% so that base isolation devices are an appropriate option. The 2.3% margin in base shear of the structure may save the extra costs when compared with a conventional un-isolated building.

Building Model	Base Floor		Middle Segment		Top Segment	
	k_o (KN/m)	F_y (KN)	k_o (KN/m)	F_y (KN)	k_o (KN/m)	F_y (KN)
Base Isolated Building	157080	471.24	----	----	----	----
Segmental Building	181710	545.13	118750	356.25	106428	319.28

(a) Structures Designed to NZS 3101:1995

Building Model	Base Floor		Middle Segment		Top Segment	
	k_o (KN/m)	F_y (KN)	k_o (KN/m)	F_y (KN)	k_o (KN/m)	F_y (KN)
Base Isolated Building	208130	1040.65	----	----	----	----
Segmental Building	240330	1201.65	159100	795.50	142966	714.83

(b) Structures Designed to NZS 3101:1982

Note :

1. The Building Models are used as discussed in Sections 3.4.3 and 3.6.
2. k_o and F_y are respectively the initial stiffnesses and yield strengths of the isolation systems.

Table 8.1 Initial Stiffnesses and Yield Strengths of Isolation Systems for Base Isolated and Segmental Buildings

However, if the client requires minimal damage in the superstructure, a segmental building makes this possible.

Step 3: From Section 4.5 and Table 4.1, the yield strength of the isolation system to the weight of the structure, F_y / W is taken as 3%.

Step 4: The base isolated model is created as shown in Appendix E.

Step 5: The initial stiffness of the isolator, k_o, is taken as ten times the total weight, W, of the structure per metre (10.0 W/m). The post-yield stiffness, αk_o, is taken as 0.4 W/m, i.e. $\alpha = 0.04$ for the bilinear model. The yield strength of the isolator, F_y, is taken from Step 3.

Step 6: The seismic responses of the base isolated building with bilinear isolators and comparisons of the un-isolated building under the scaled to the El Centro 1940 N-S earthquake are computed in a manner similar to those shown in Chapter 7.

Step 7: The isolator displacement is 0.097 metre. This is within the displacement limits of most isolators. If the isolator displacement is greater than the displacement limit of the isolator, Step 5 should be repeated.

Step 8: Design reinforcement details of the members.

b. Procedure for the Design of the Segmental Building

Steps 1 to 3 are the same as the design of the base isolated building.

Step 4: When the segmental building was divided into three segments as shown in Fig. 3.13 of Chapter 3, the segmental model was created as shown in Appendix E.

Step 5: The post-yield stiffness, αk_o, is taken as 0.5 W/m, i.e. $\alpha = 0.05$ for the bilinear model. The initial stiffness and yield strength of the isolation system are used as follow:

Base Floor		Middle Segment		Top Segment	
k_o (KN/m)	F_y (KN)	k_o (KN/m)	F_y (KN)	k_o (KN/m)	F_y (KN)
240330	720.99	159100	477.30	142966	428.90

Step 6: The seismic responses of the segmental building with bilinear isolators and comparisons of the un-isolated building under the scaled to the El Centro 1940 N-S earthquake are computed in a manner similar to those shown in Chapter 7.

Step 7: The isolator displacements of the top segment, middle segment and base floor are 0.054, 0.066 and 0.043 metres respectively. This is within the displacement limits of most isolators. The maximum displacement between the two segments is 0.066 metre whereas the code 2.5 % interstorey drift limit for a storey height of 3.65 metres is 0.091 metre. So, this means that the isolators could be placed within the storey. If the isolator displacement is greater than the displacement limit of the isolator, Step 5 should be repeated.

Step 8: Design reinforcement details of the members.

8.4 Summary and Conclusion

In this chapter, a basic estimate for the analysis and design of the base isolated and segmental structures is presented for use in the preliminary design stage. These preliminary design choices form the basis of the final design stage, which needs further optimization of the aseismic design based on more accurate evaluations of seismic responses, and for a more detailed design of the structure and isolation devices.

The base isolated and segmental buildings have been properly designed to resist wind load, so that the isolation devices are designed to have nonlinear hysteretic characteristics such as elasto-plastic or bilinear hysteresis. In this study, the yield force of the isolation system to the weight of the structure was determined by comparison with the base shears from earthquake-induced and wind-induced loads of a 12-storey multistorey building based on the equivalent static method suggested by NZS 4203:1992 as discussed in Section 4.3.

CHAPTER 9

SUMMARY AND CONCLUSIONS

9.1 Summary

The seismic responses of the long period (greater than 2 seconds) multistorey base isolated and segmental buildings using nonlinear isolation systems under the different types of ground motions have been studied in this thesis. The seismic isolator was simulated by an elasto-plastic or bilinear hysteretic model for the time history analyses. The main parameters are the dynamic properties of the isolator including initial stiffness, post-yield stiffness and yield strength.

In Table 4.1 of Chapter 4, the structures designed to NZS 3101:1982 [N1] and NZS 3101:1995 [S9] were used to compute the base shear of a structure induced by an earthquake using the equivalent static method and these were compared with that induced by wind loading based on wind forces acting on buildings as specified by NZS 4203:1992. The wind forces are usually treated as an elastic response whereas the earthquake actions are usually resisted in a ductile inelastic response. The isolator yield force is determined by the level of the wind loading used in design with a margin to prevent yield under design wind-storm conditions. Therefore, it is required that the base shear from earthquake-induced forces must be greater than the base shear from wind-induced forces if base isolation devices are to be used for seismic design.

In Sections 4.5 and 5.2, an optimum value of the initial stiffness of the isolator, k_o, was found to be ten times the total weight, W, of the structure per metre (10.0 W/m). The post-yield stiffnesses, αk_o, were found to be 0.4 and 0.5 W/m, i.e. $\alpha = 0.04$ and 0.05 for the bilinear models in the base isolated and segmental structures respectively. The yield strength of the base isolator, F_y, was found to be a value of between 3% to 5% W.

From Chapter 5, it was found that the base isolated and segmental buildings on a compliant foundation have a longer fundamental period when compared with the buildings on a rigid foundation. This is because the flexibility of the foundation increases the fundamental period of free vibration of the structure. These period shifts are significantly affected by the soil stiffness. This shift in the period affects the structural responses. Therefore, the effect of foundation compliance on the protection provided by the base isolation system was investigated and compared with the case of a rigid base foundation..

From the equivalent static method of NZS 4203:1992 [S8], the maximum applied force is based on the horizontal seismic shear force acting at the base of the structure in the direction of the earthquake excitation. Therefore, an additional equivalent viscous damping was calculated based on the time history analyses using a harmonically varying load pattern. The structures with additional damping ratios having elasto-plastic and bilinear isolators obtained an equivalent extra viscous damping of 8% and 15% respectively. The effects of the additional damping on the seismic responses of the structures were investigated in Chapter 6.

For the structures with and without velocity-dependent viscous damping devices when designed to NZS 3101:1995 as shown in Chapters 5 and 6, the base shears of the fixed base buildings using the time history analyses under the El Centro 1940 N-S earthquake are respectively 26% and 38% greater than those of the fixed base buildings from the equivalent static method of NZS 4203:1992. Similar results are also obtained for the structures designed to NZS 3101:1982. This means that the El Centro 1940 N-S excitation used in the analyses is more severe than the design earthquake implied in NZS 4203:1992.

To compare the base shear forces in the structures subjected to the earthquake with those implied by the loading code, the earthquake accelergrams were scaled. In Chapter 7, a weighted scale factor for the fundamental, second and third modal periods of the structures multiplying the El Centro 1940 N-S earthquake acceleration by 1.07 and 1.29 respectively for the fixed base buildings designed to NZS 3101:1995 and NZS 3101:1982 was used. For the structures designed to NZS 3101:1995, the base shear of the fixed base building using the time history analyses under the El Centro 1940 N-S earthquake is 30% greater than the base shear obtained from the equivalent static method of NZS 4203:1992. However, a good agreement (less than 5% variation) is obtained for the structures designed to NZS 3101:1982.

When an average of the scaled factors for the fundamental periods from two to three second range of the structure was used, resulting in multiplying the El Centro 1940 N-S earthquake acceleration by 1.27, the base shear of the fixed base building designed to NZS

3101:1995 using the time history analyses is approximately 10% greater than the base shear obtained from the equivalent static method of NZS 4203:1992. So, the scale factor, 1.27 would have been much more appropriate than the scale factor, 1.07 used earlier. Therefore, scaling the El Centro 1940 N-S earthquake by an average of the scaled factors for the fundamental periods from two to three second range of the structure appears to be better than scaling by a weighted scale factor for the fundamental, second and third modal periods of the structures

9.2 Conclusions

1. The benefits of implementing a seismic isolation system were investigated by comparing the performance of base isolated, segmental and fixed base multistorey buildings. With the inclusion of seismic isolation devices, the base isolated and segmental buildings with elasto-plastic and bilinear isolation devices have significantly reduced top floor deflections, accelerations, interstorey drifts and base shears when compared with the fixed base building.

2. From a strictly strength viewpoint, the base isolated building does not show a great reduction in base shear when compared with the conventional fixed base building. If a smaller lateral storey displacement is required in the design of a multistorey structure without auxiliary damping mechanisms, the base isolated and segmental buildings could be used. If a very small interstorey drift (less than 0.5%) in a structure is required, it would be more appropriate to use a segmental building. If the lateral storey displacements are required to be further reduced, the installation of velocity-dependent viscous dampers in a structure should be considered.

3. For the long period (greater than 2 seconds) base isolated structures during the time history analyses undertaken, the post-yield stiffness ratio of the bilinear isolator was found to be 0.04 which is significantly less than the value of 0.15 used by Andriono (1990). Andriono used structures with short fundamental periods (less than 1.0 second). For the segmental building, the post-yield stiffness ratio of the bilinear isolator of 0.05 was found to be an optimum value.

4. The segmental buildings possess the ability to decouple the building from the earthquake ground motions in a manner similar to that of the base isolated building. While keeping the ratio of yielding force of the isolation system to weight of structure low, a segmental building significantly reduces the base displacement response when compared with that of a base isolated building. This solves the problems associated with the large displacements required by the base isolators in buildings under some excitations such as near-fault earthquake and which displacement may not available in many current isolation devices.

5. The base isolated and segmental buildings have significantly reduced maximum beam curvature ductility demands (near to 1.0) when compared with the fixed base building. This implys that the design of structural members for the base isolated and segmental buildings may not need to be necessarily designed to the full ductility requirements of the code. Therefore, this isolation technique widens the choice of architectural forms and structural materials as well as reduces the structural and non-structural damage.

6. The segmental building concept can be considered as an extension of the base isolation technique with a distributed flexibility in the superstructure. Absorption and dissipation of earthquake energy are provided by all isolators in the segmental building, rather than only by the isolation system at the base level. In this study, each isolator displacement in the segmental building is smaller than the base isolator displacement in a base isolated structure.

7. When compared with the fixed base building, there are significant reductions of base shears and interstorey drifts for the base isolated and segmental buildings with elasto-plastic and bilinear isolation systems under the Mexico 1985 SCT earthquake. For sites with ground motions which have peak spectral acceleration in longer periods, such as the Mexico 1985 SCT earthquake, the inclusion of seismic isolation devices may shift the fundamental periods of the structures beyond the region of the peak spectral accelerations into the descending part of the spectrum for longer period (greater than 2 seconds) structures. It appears that these base isolated structures are not disadvantaged by this type of excitation.

8. With the inclusion of extra velocity-dependent damping devices, the fixed base, base isolated and segmental structures have significantly reduced top floor deflections and interstorey drifts when compared with the same buildings without additional damping. The total base shears are similar in both structural damping configurations but the shears in the structural members are reduced.

9. The lateral displacements in the upper parts of the base isolated and segmental buildings with foundation compliance show greater differences when compared with the same buildings with a rigid base. This is because of the contributions from rocking of the structures with foundation compliance. Therefore, the effects of rocking need to be considered in the design of long period base isolated and segmental buildings with foundation compliance. Similar effects would be observed in a fixed base building with foundation compliance.

10. The main purpose of scaled earthquake records is to match the requirements of design spectrum of NZS 4203:1992 when the time history analysis is used for analysing the ultimate limit state building responses. Therefore, the earthquake records were scaled according to their 5% damped spectra to match the design spectrum in Section 4.6.2.9 (b) (ii) of NZS 4203:1992 for the intermediate soil sites.

    The method based on weighted scale factors for the fundamental (weight 2.0), and second and third (weight 1.0) modal periods of the structures was used to compute the scale factors for the three short period earthquake records chosen. For a long period excitation, scaling at the fundamental period of the structure was used.

    In this study, the contributions of higher modes of the structure were considered when choosing a scale factor for the long period structures. For these structures, the fundamental, second and third modal periods of the structure are approximately 2.5, 0.9 and 0.5 seconds respectively. From the spectra of the short period ground motions chosen, the peak spectral accelerations occur for natural periods less than 1.0 second, and the structural responses may be affected by the contributions of higher modes of the structure. For the spectrum of a long period excitation such as Mexico 1985 SCT S00E, the peak spectral acceleration occurs at about 2.0 seconds. The earlier weighted scaling method gave structural responses markedly increased by the contributions of the higher modes of the structure as discussed in Section 7.2.2. Therefore, the scaling should only consider the effect of the fundamental mode of the structure for this long period excitation.

9.3 Recommendations for Further Research

Some important results and findings of the seismic performance and design of base isolated and segmental multistorey structures have been highlighted. However, further research of the following issues is necessary in order to be able to establish a complete design philosophy for these types of structures.

1. In this study, the soil shear stiffness was assumed to be constant throughout the duration of the excitation which implies a constant soil shear modulus and linear soil-foundation impedances. Further research is required to consider a time dependent effect and displacement dependent effect on the soil shear modulus.

2. The dynamic soil-foundation impedances used are assumed to possess constant coefficients and the analyses are frequency independent. From the discussion in many references, the dynamic effects are frequency dependent and result in soil-foundation impedances not only varying with time but also with frequency. A more precise analysis is required to take into account these variations and to determine these effects when used in nonlinear time-domain analyses.

3. Further research is required to investigate the effect of equipment mounted on the floors of the base isolated and segmental structures for various types of isolators.

4. Further research is required to investigate the effects of base isolation systems on the seismic response of multistorey structures with inelastic superstructures. The results are expected to clarify the effectiveness of base isolators in protecting

buildings with soft-storeys and to give more guidance in designing the members of the superstructure and their detailing without requiring fully ductile design.

5. In this study, the initial stiffness and yield strength of the isolation system were considered according to the supported weights of segments for the segmental building. Further research is required to consider the variations of the initial stiffness and yield strength of the isolation system and to investigate the effects on the segmental structure during the inelastic time history analyses.

6. A displacement-dependent base isolation system may reduce the first mode storey shears significantly but generally do not reduce the higher mode storey shears to the same degree. This is true for both the base isolated and segmental buildings. It is very desirable that further research of an isolation device with an elastic spring and a velocity-dependent damper, which may tend to suppress the higher mode effects, should be carried out.

7. Further research is required to consider a three-dimensional modelling of the base isolated and segmental buildings mounted on various types of isolation systems under concurrent orthogonal seismic excitations and then to investigate the effects of different isolation systems on the torsional responses. This includes the investigation of the torsional effects of unsymmetric base isolated and segmental multistorey structures with various types of isolation devices.

8. In this study, the earthquake records were scaled according to their 5% damped spectra to match the design spectrum of NZS 4203:1992. Further research is required to use different possibilities for scaling earthquake records, e.g. scaled to the peak ground acceleration, to investigate the structural responses of the base isolated and segmental buildings with various types of isolation systems.

9. In this study, only framed buildings were used to investigate the seismic responses of the base isolated and segmental buildings with various types of isolators. Further research is required to consider a greater variation of the structural forms such as wall and wall-frame structures.

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APPENDIX A

SEISMIC PERFORMANCES OF STRUCTURES DESIGNED TO NZS 3101:1982 UNDER THE El CENTRO 1940 N-S EARTHQUAKE

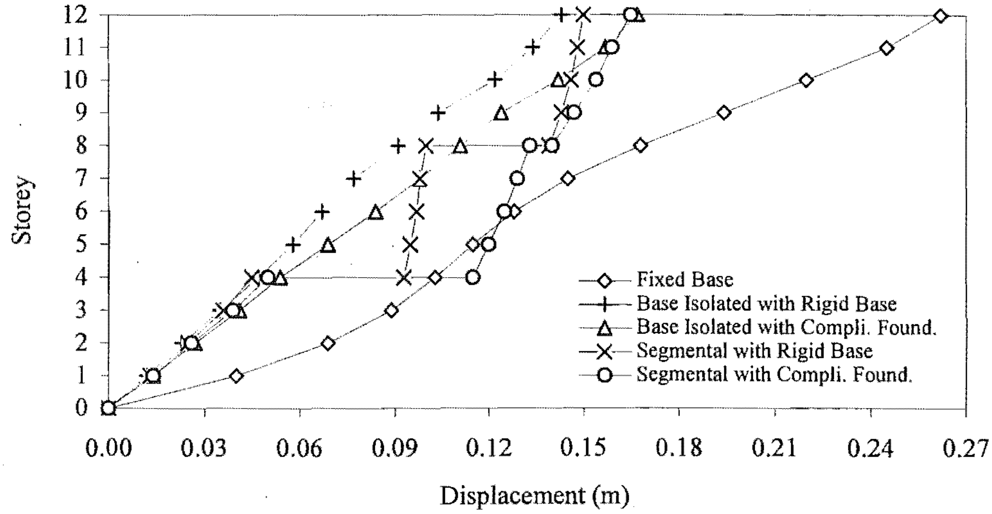
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.47%	0.25%	0.27%	0.08%	0.16%
11	0.68%	0.33%	0.41%	0.08%	0.14%
10	0.71%	0.49%	0.49%	0.08%	0.19%
9	0.71%	0.36%	0.36%	0.11%	0.19%
8	0.63%	0.38%	0.36%	0.08%	0.11%
7	0.47%	0.27%	0.38%	0.08%	0.11%
6	0.36%	0.25%	0.41%	0.08%	0.14%
5	0.33%	0.30%	0.41%	0.08%	0.14%
4	0.38%	0.33%	0.36%	0.25%	0.30%
3	0.55%	0.33%	0.38%	0.30%	0.36%
2	0.79%	0.30%	0.36%	0.33%	0.33%
1	0.80%	0.24%	0.28%	0.26%	0.28%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

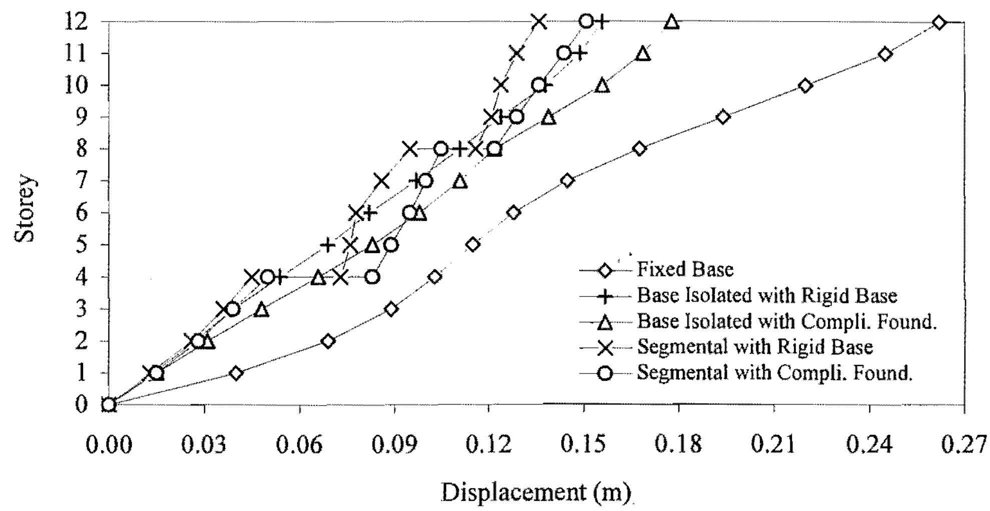
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.47%	0.19%	0.25%	0.19%	0.19%
11	0.68%	0.30%	0.36%	0.14%	0.22%
10	0.71%	0.38%	0.47%	0.11%	0.19%
9	0.71%	0.36%	0.47%	0.14%	0.19%
8	0.63%	0.38%	0.30%	0.25%	0.14%
7	0.47%	0.41%	0.36%	0.22%	0.14%
6	0.36%	0.36%	0.41%	0.11%	0.16%
5	0.33%	0.41%	0.47%	0.11%	0.16%
4	0.38%	0.41%	0.49%	0.25%	0.30%
3	0.55%	0.33%	0.47%	0.27%	0.30%
2	0.79%	0.36%	0.44%	0.36%	0.30%
1	0.80%	0.28%	0.30%	0.26%	0.30%

(b) Structures Mounted on Bilinear Isolation Systems

Table A.1 Interstorey Drifts for Different Structures



(a) Elasto-Plastic Model of the Isolation Systems

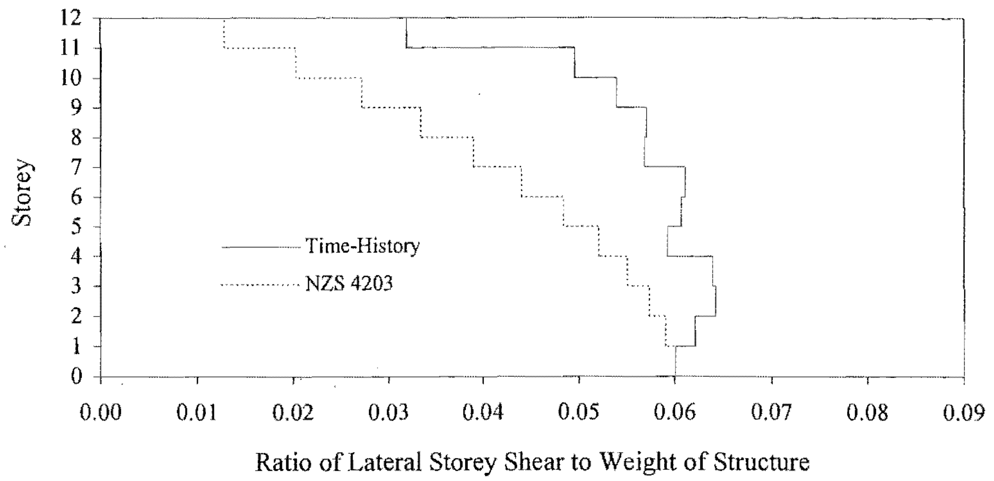


(b) Bilinear Model of the Isolation Systems

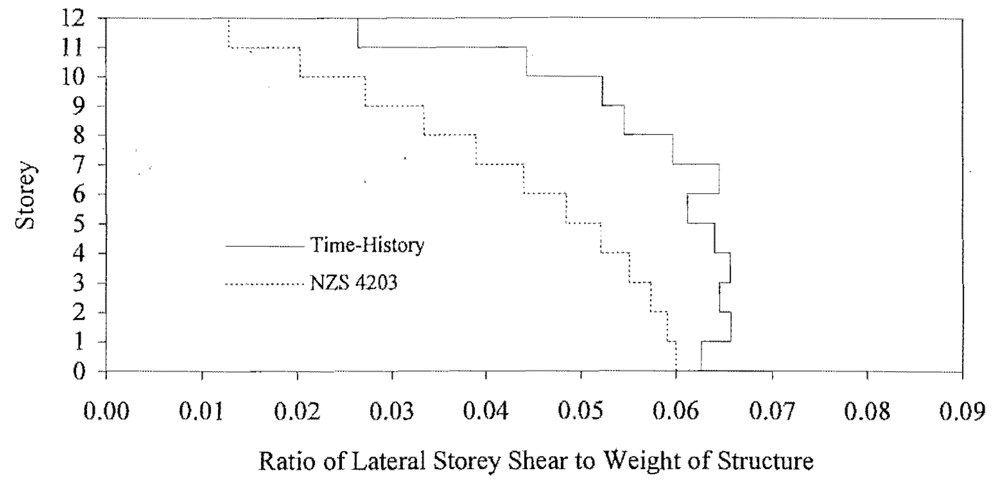
Fig. A.1 Comparisons of Displacement with Storey for Fixed Base, Base Isolated and Segmental Buildings

Type	Base Shear / Total Weight of Structure		
	Time History Analysis		Equivalent Static Method of NZS 4203:1992
	Elasto-Plastic Model	Bilinear Model	
Fixed Base Building	0.0812		0.0600
Base Isolated Building on a Rigid Base	0.0601	0.0681	0.0600
Base Isolated Building on a Compliant Foundation	0.0626	0.0665	0.0600
Segmental Building on a Rigid Base	0.0575	0.0591	0.0600
Segmental Building on a Compliant Foundation	0.0594	0.0620	0.0600

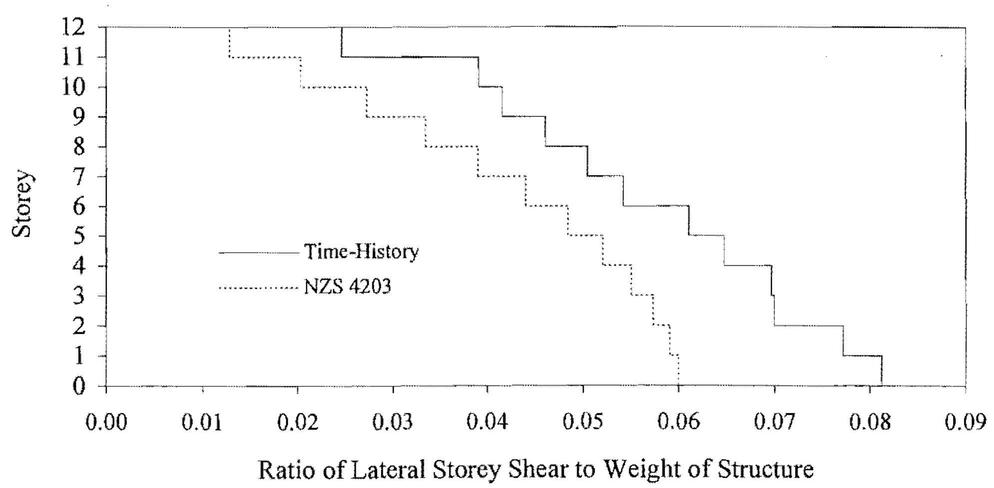
Table A.2 Normalised Base Shears for Different Buildings



(a) Base Isolated Building with Rigid Base

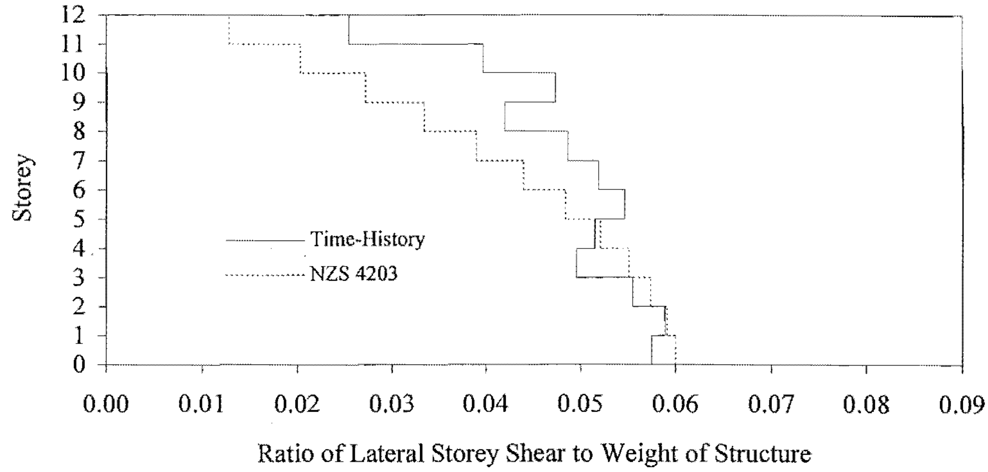


(b) Base Isolated Building with Foundation Compliance

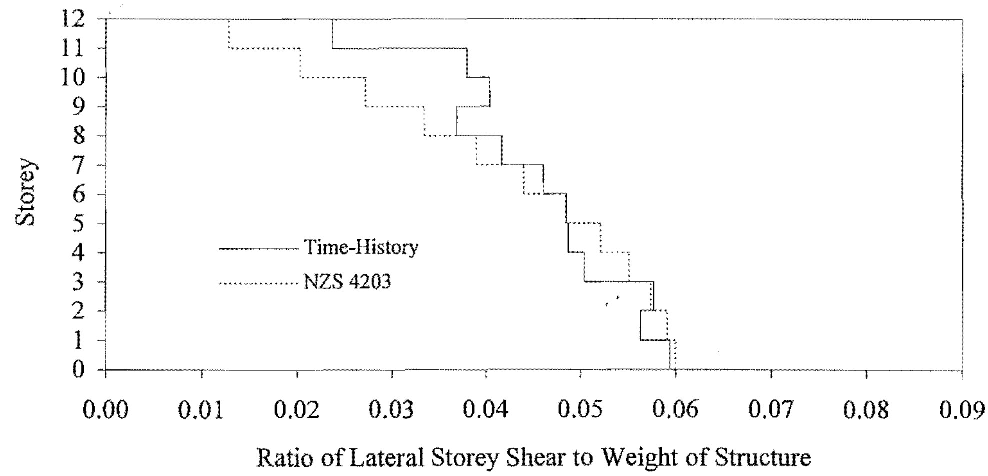


(c) Fixed Base Building

Fig. A.2 Lateral Storey Shear Envelopes for Structures with Elasto-Plastic Isolation Systems

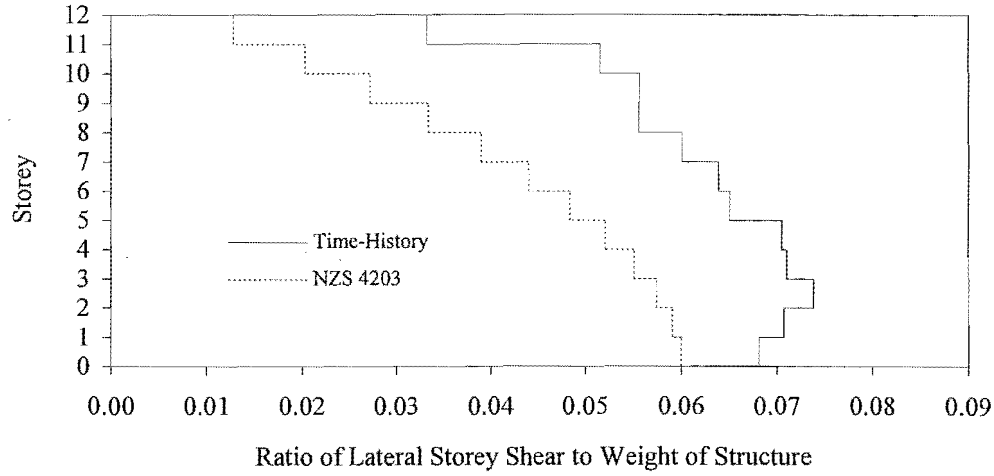


(d) Segmental Building with Rigid Base

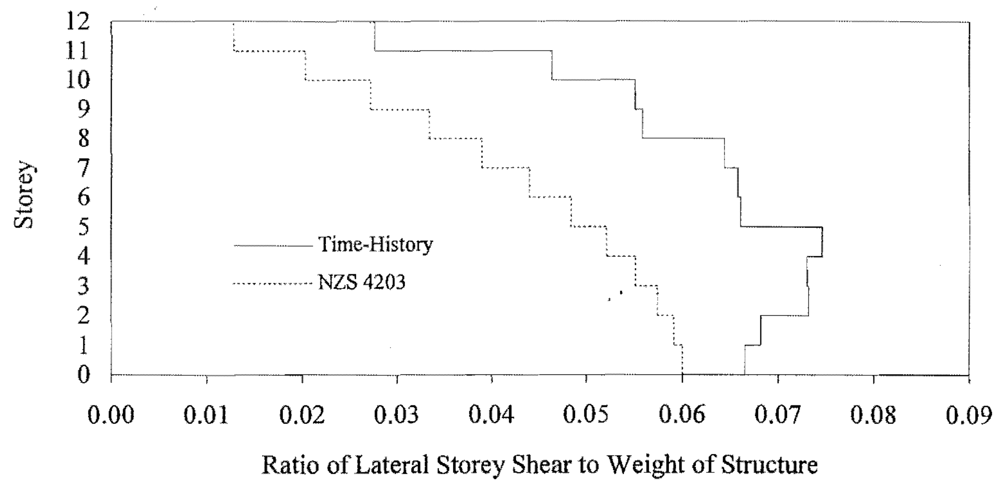


(e) Segmental Building with Foundation Compliance

Fig. A.2 ..... (continued)

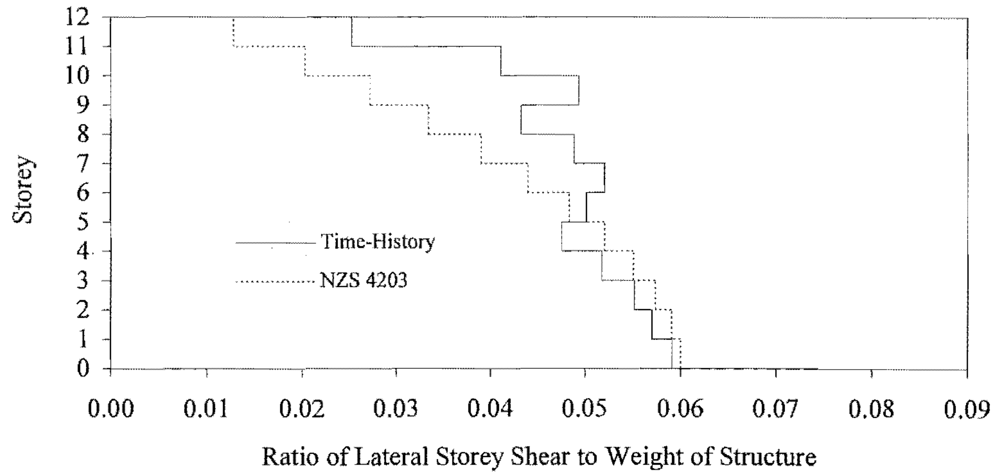


(a) Base Isolated Building with Rigid Base

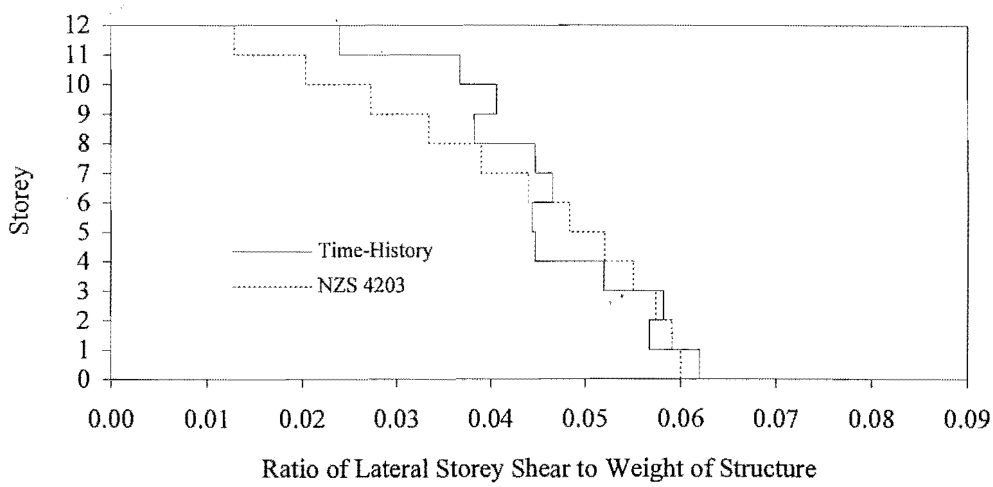


(b) Base Isolated Building with Foundation Compliance

Fig. A.3 Lateral Storey Shear Envelopes for Structures with Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation Compliance

Fig. A.3 ..... (continued)

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	7.51	2.35	1.63	2.85	2.38
	11	11.88	3.10	2.88	2.98	3.44
	10	10.47	2.44	1.48	1.42	1.07
	9	10.80	3.12	2.28	1.62	1.19
	8	5.95	1.95	1.81	< 1.00	< 1.00
	7	5.59	1.94	1.97	1.18	1.06
	6	4.47	1.31	1.10	< 1.00	< 1.00
	5	4.24	< 1.00	1.06	< 1.00	< 1.00
	4	3.06	1.03	1.25	< 1.00	< 1.00
	3	3.48	1.14	1.15	< 1.00	< 1.00
	2	4.59	1.13	1.23	1.19	1.05
	1	7.12	1.08	1.12	1.28	1.26
Column Bases	L.Ext.*	2.11	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.13	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	7.51	2.52	1.82	2.48	2.57
	11	11.88	3.12	2.85	3.29	3.51
	10	10.47	2.28	1.51	1.55	1.38
	9	10.80	3.28	2.51	1.24	1.33
	8	5.95	2.15	2.18	< 1.00	< 1.00
	7	5.59	2.34	2.50	1.02	1.07
	6	4.47	1.28	1.28	< 1.00	< 1.00
	5	4.24	1.63	1.47	< 1.00	< 1.00
	4	3.06	2.05	1.79	< 1.00	< 1.00
	3	3.48	1.92	1.92	< 1.00	< 1.00
	2	4.59	1.61	1.83	1.11	1.08
	1	7.12	1.45	1.33	1.37	1.24
Column Bases	L.Ext.*	2.11	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	2.13	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table A.3 Maximum Curvature Ductility Demands for Different Structures

APPENDIX B

SEISMIC PERFORMANCES OF STRUCTURES WITH ADDITIONAL DAMPING WHEN DESIGNED TO NZS 3101:1982 UNDER THE EL CENTRO 1940 N-S EARTHQUAKE

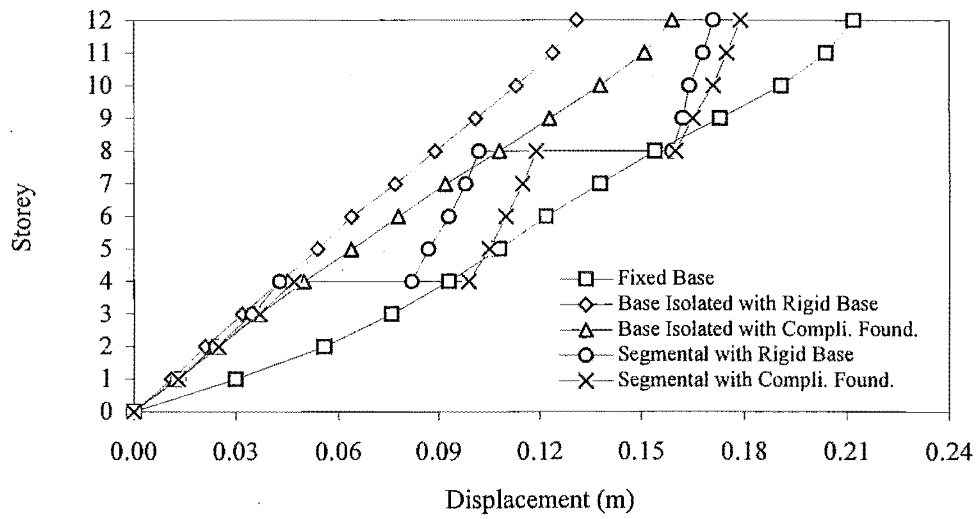
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.22%	0.19%	0.22%	0.08%	0.11%
11	0.36%	0.30%	0.36%	0.11%	0.11%
10	0.49%	0.33%	0.41%	0.08%	0.16%
9	0.52%	0.33%	0.41%	0.08%	0.14%
8	0.44%	0.33%	0.44%	0.11%	0.11%
7	0.44%	0.36%	0.38%	0.14%	0.14%
6	0.38%	0.27%	0.38%	0.16%	0.14%
5	0.41%	0.30%	0.38%	0.14%	0.16%
4	0.47%	0.30%	0.36%	0.22%	0.27%
3	0.55%	0.30%	0.36%	0.30%	0.33%
2	0.71%	0.27%	0.33%	0.30%	0.33%
1	0.60%	0.22%	0.24%	0.26%	0.26%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

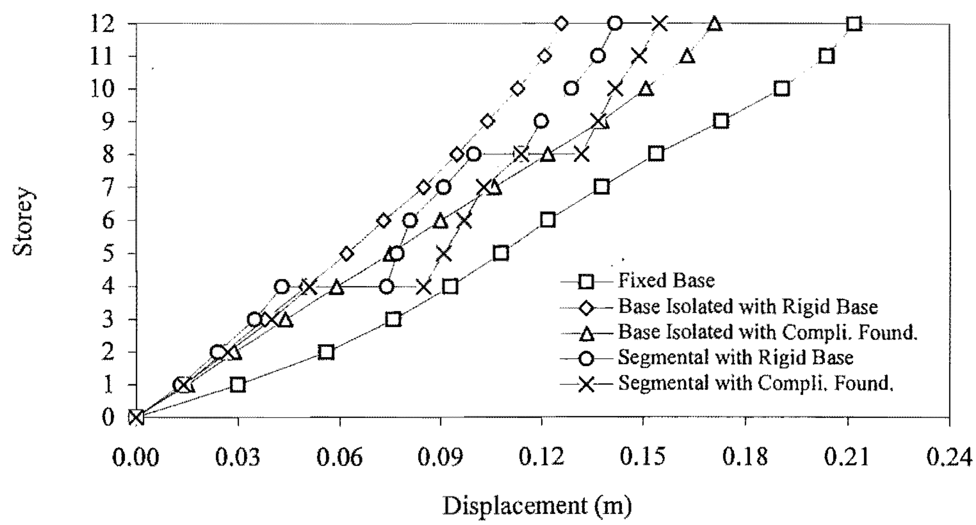
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.22%	0.14%	0.22%	0.14%	0.16%
11	0.36%	0.22%	0.33%	0.22%	0.19%
10	0.49%	0.25%	0.36%	0.25%	0.14%
9	0.52%	0.25%	0.44%	0.16%	0.14%
8	0.44%	0.27%	0.44%	0.25%	0.30%
7	0.44%	0.33%	0.44%	0.27%	0.16%
6	0.38%	0.30%	0.41%	0.11%	0.16%
5	0.41%	0.33%	0.44%	0.11%	0.16%
4	0.47%	0.33%	0.41%	0.22%	0.30%
3	0.55%	0.33%	0.41%	0.30%	0.36%
2	0.71%	0.33%	0.38%	0.30%	0.36%
1	0.60%	0.28%	0.30%	0.26%	0.28%

(b) Structures Mounted on Bilinear Isolation Systems

Table B.1 Interstorey Drifts for Different Structures with Additional Damping



(a) Elasto-Plastic Model of the Isolation Systems

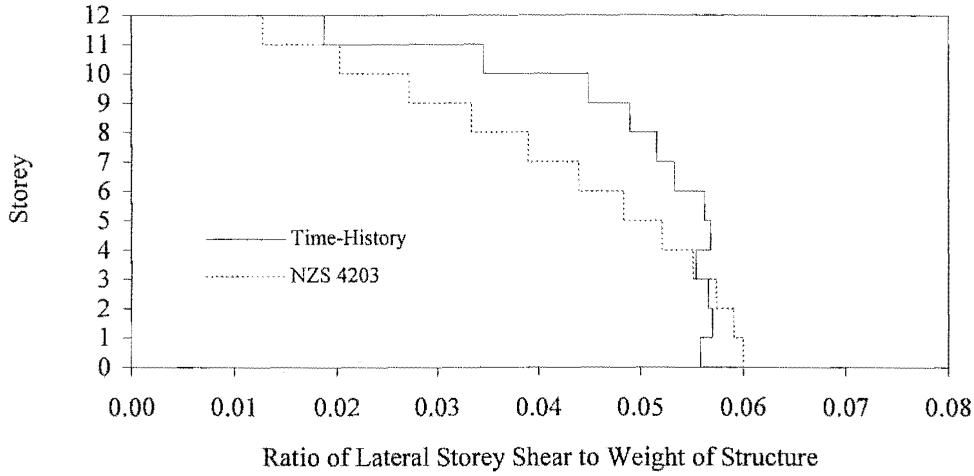


(b) Bilinear Model of the Isolation Systems

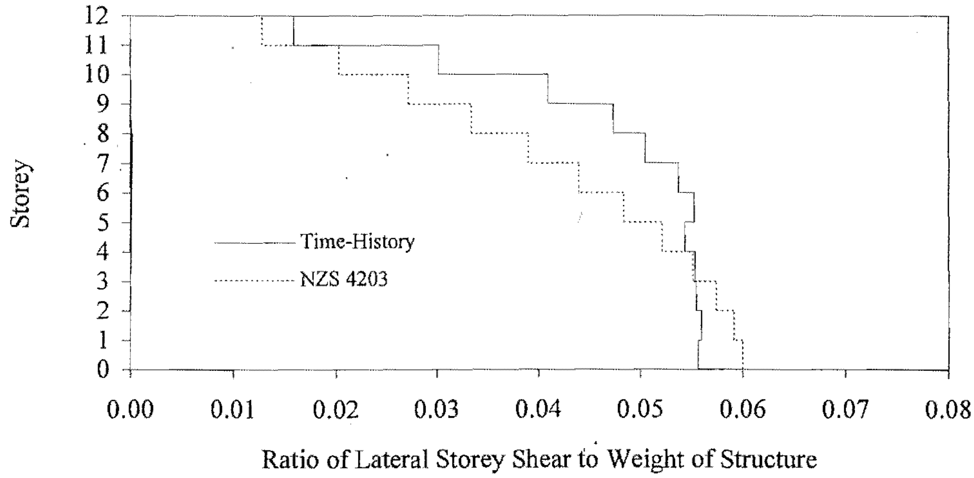
Fig. B.1 Comparison of Displacement with Storey for Fixed Base, Base Isolated and Segmental Buildings with Additional Damping

Type	Base Shear / Total Weight of Structure		
	Time History Analysis		Equivalent Static Method of NZS 4203:1992
	Elasto-Plastic Model	Bilinear Model	
Fixed Base Building	0.0768		0.0600
Base Isolated Building on a Rigid Base	0.0558	0.0687	0.0600
Base Isolated Building on a Compliant Foundation	0.0556	0.0671	0.0600
Segmental Building on a Rigid Base	0.0559	0.0562	0.0600
Segmental Building on a Compliant Foundation	0.0529	0.0573	0.0600

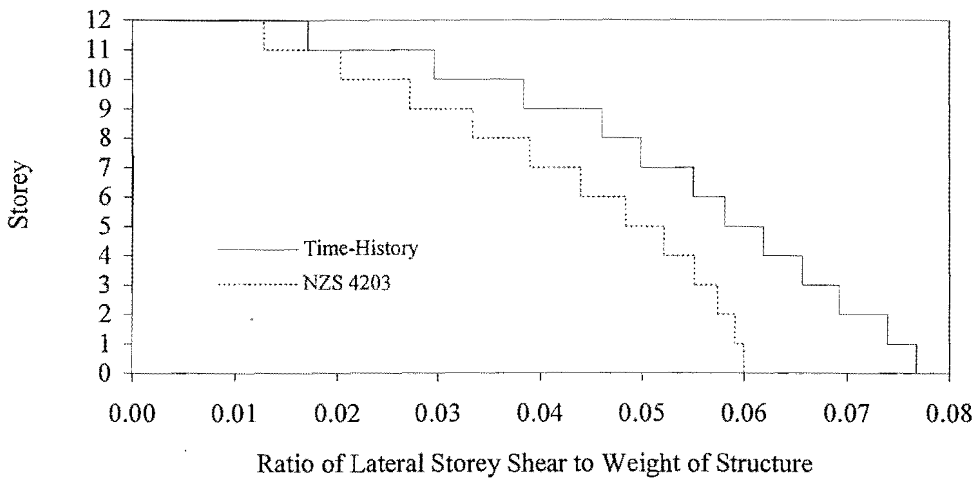
Table B.2 Normalised Base Shears for Different Buildings with Additional Damping



(a) Base Isolated Building with Rigid Base

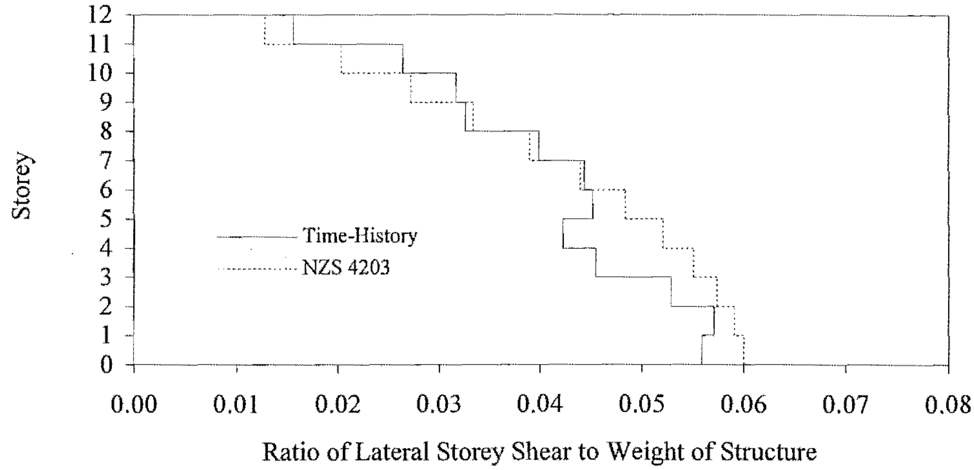


(b) Base Isolated Building with Foundation Compliance

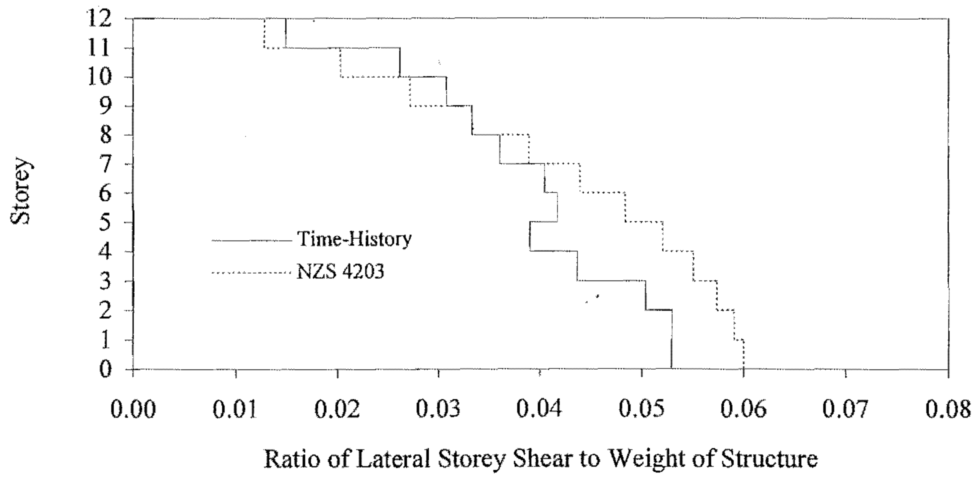


(c) Fixed Base Building

Fig. B.2 Lateral Storey Shear Envelopes for Structures with Additional Damping Using Elasto-Plastic Isolation Systems

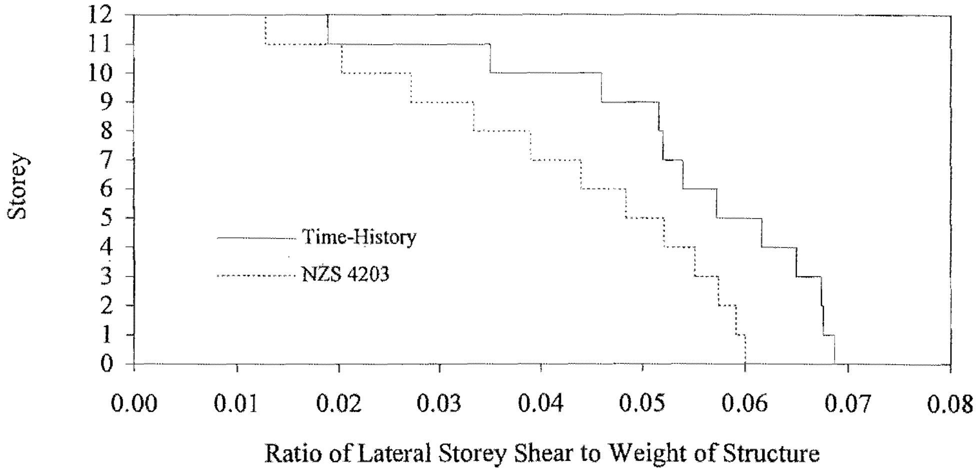


(d) Segmental Building with Rigid Base

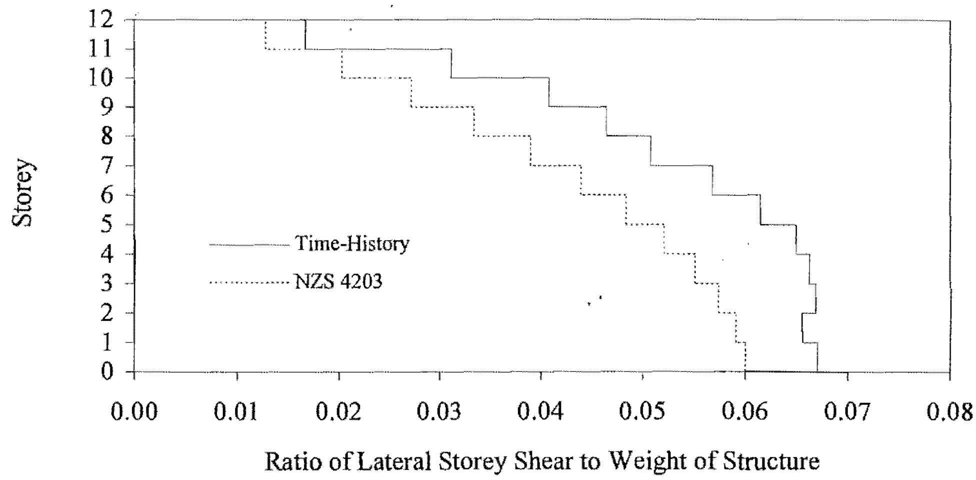


(e) Segmental Building with Foundation Compliance

Fig. B.2 ..... (continued)

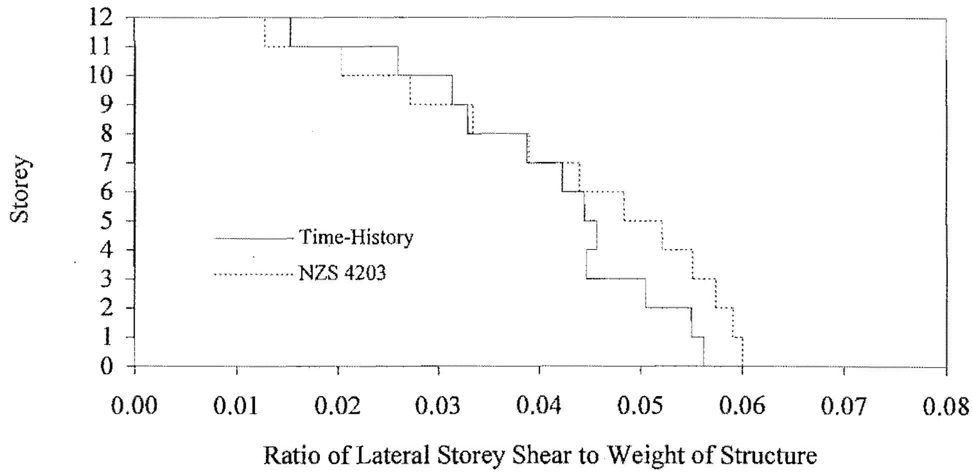


(a) Base Isolated Building with Rigid base

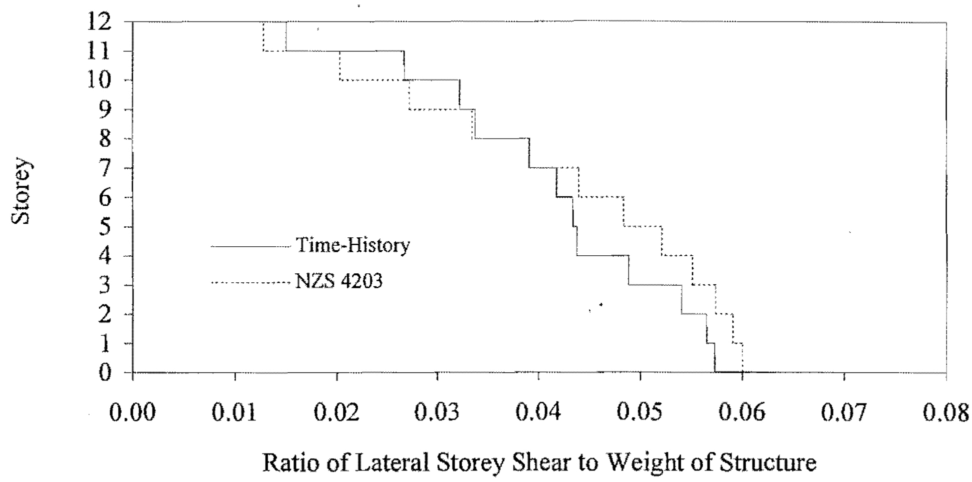


(b) Base Isolated Building with Foundation Compliance

Fig. B.3 Lateral Storey Shear Envelopes for Structures with Additional Damping Using Bilinear Isolation Systems



(c) Segmental Building with Rigid Base



(d) Segmental Building with Foundation Compliance

Fig. B.3 ..... (continued)

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	3.01	< 1.00	< 1.00	< 1.00	< 1.00
	11	8.52	< 1.00	1.51	1.17	1.67
	10	5.91	< 1.00	< 1.00	< 1.00	< 1.00
	9	7.43	1.52	1.10	< 1.00	< 1.00
	8	4.21	< 1.00	1.04	< 1.00	< 1.00
	7	4.29	< 1.00	1.08	< 1.00	< 1.00
	6	3.62	< 1.00	< 1.00	< 1.00	< 1.00
	5	3.73	< 1.00	< 1.00	< 1.00	< 1.00
	4	2.93	< 1.00	< 1.00	< 1.00	< 1.00
	3	3.32	< 1.00	< 1.00	< 1.00	< 1.00
	2	4.27	< 1.00	< 1.00	1.05	< 1.00
	1	5.81	< 1.00	< 1.00	1.17	< 1.00
Column Bases	L.Ext.*	1.41	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.44	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	3.01	< 1.00	< 1.00	< 1.00	< 1.00
	11	8.52	< 1.00	1.56	1.42	1.76
	10	5.91	< 1.00	< 1.00	< 1.00	< 1.00
	9	7.43	1.13	< 1.00	< 1.00	< 1.00
	8	4.21	< 1.00	1.02	< 1.00	< 1.00
	7	4.29	< 1.00	1.21	< 1.00	< 1.00
	6	3.62	< 1.00	< 1.00	< 1.00	< 1.00
	5	3.73	< 1.00	1.08	< 1.00	< 1.00
	4	2.93	1.09	1.25	< 1.00	< 1.00
	3	3.32	1.24	1.27	< 1.00	< 1.00
	2	4.27	1.25	1.32	< 1.00	1.10
	1	5.81	1.43	1.16	1.20	1.21
Column Bases	L.Ext.*	1.41	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.44	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table B.3 Maximum Curvature Ductility Demands for Different Buildings with Added Damping

APPENDIX C

OVERALL RESPONSE QUANTITIES OF STRUCTURES DESIGNED TO NZS 3101:1982 UNDER THE FOUR SCALED EARTHQUAKE RECORDS

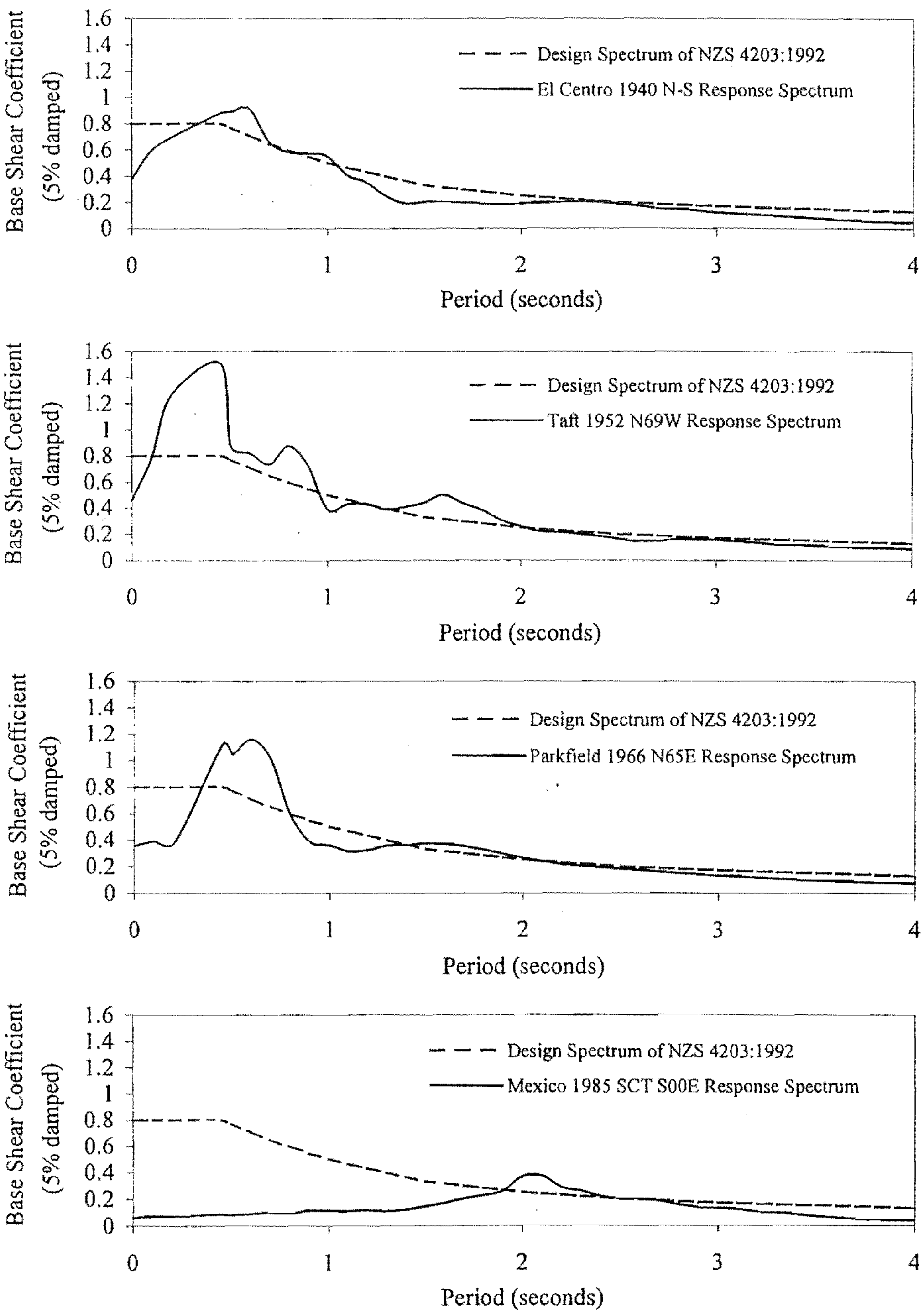


Fig. C.1 Base Shear Coefficients for the Earthquake Response Spectrum and Design Spectrum for Fixed base and Base Isolated Buildings Designed to NZS 3101:1995

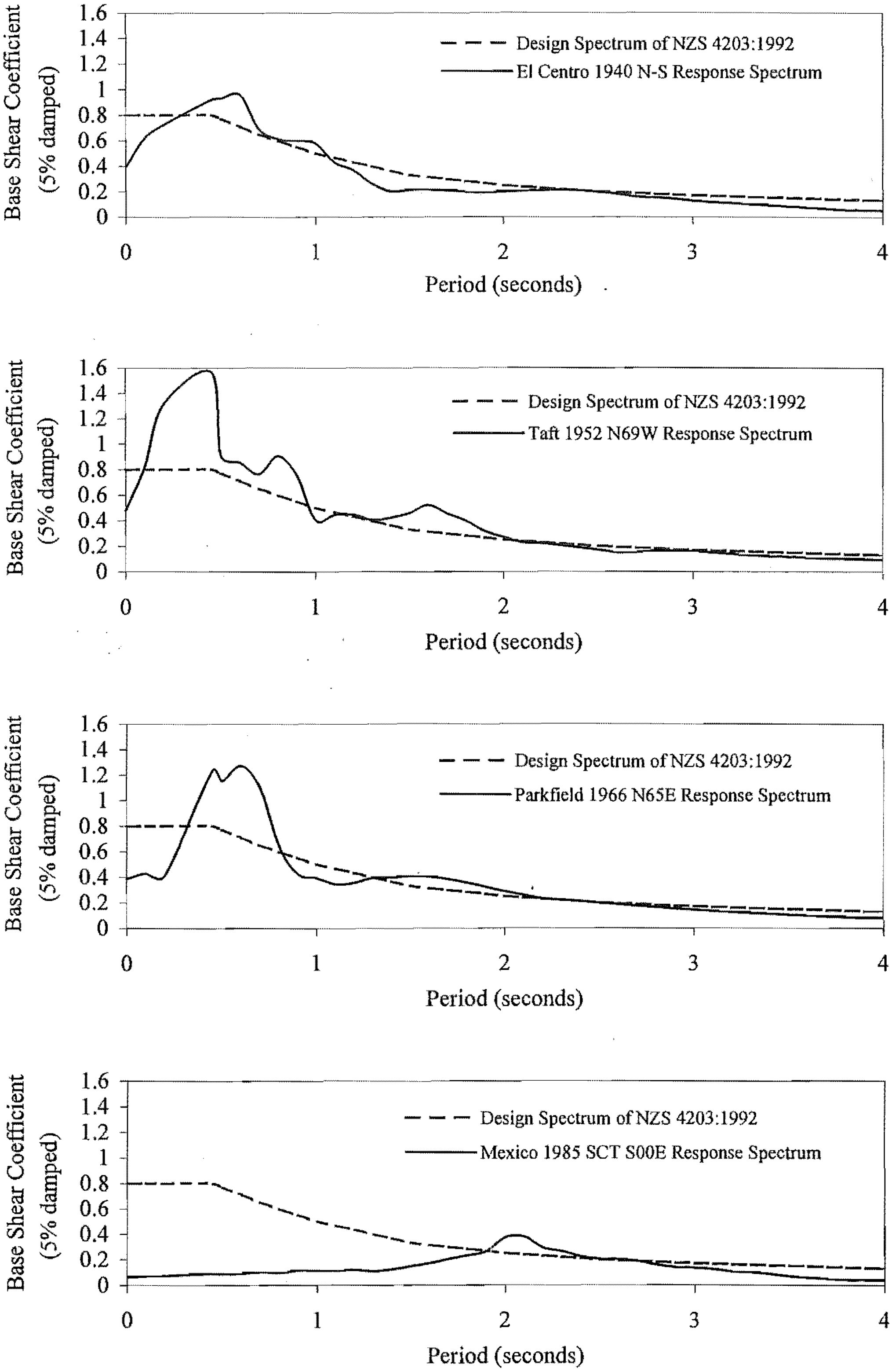


Fig. C.2 Base Shear Coefficients for the Earthquake Response Spectrum and Design Spectrum for Segmental Buildings Designed to NZS 3101:1995

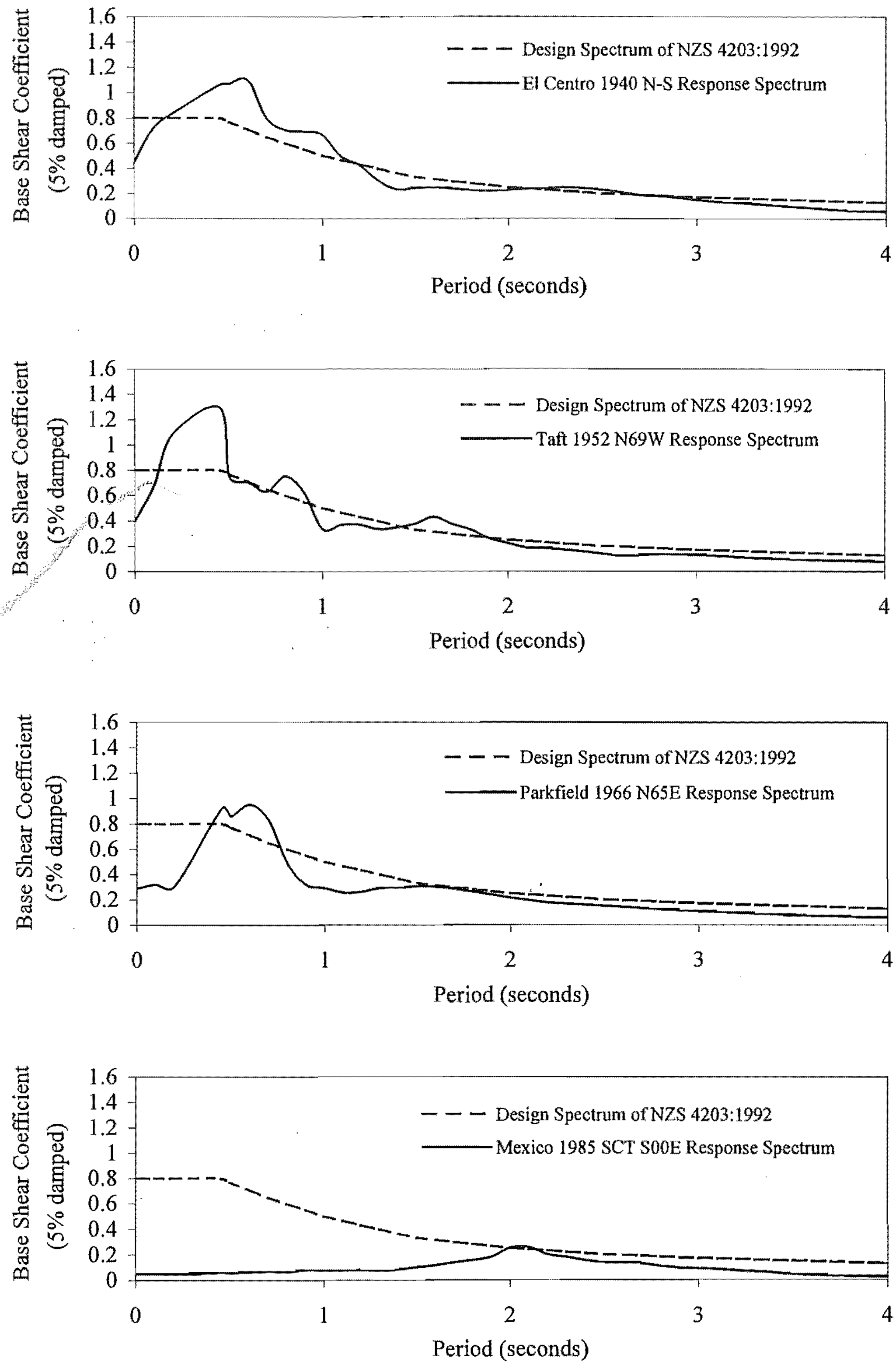


Fig. C.3 Base Shear Coefficients for the Earthquake Response Spectrum and Design Spectrum for Fixed base and Base Isolated Buildings Designed to NZS 3101:1982

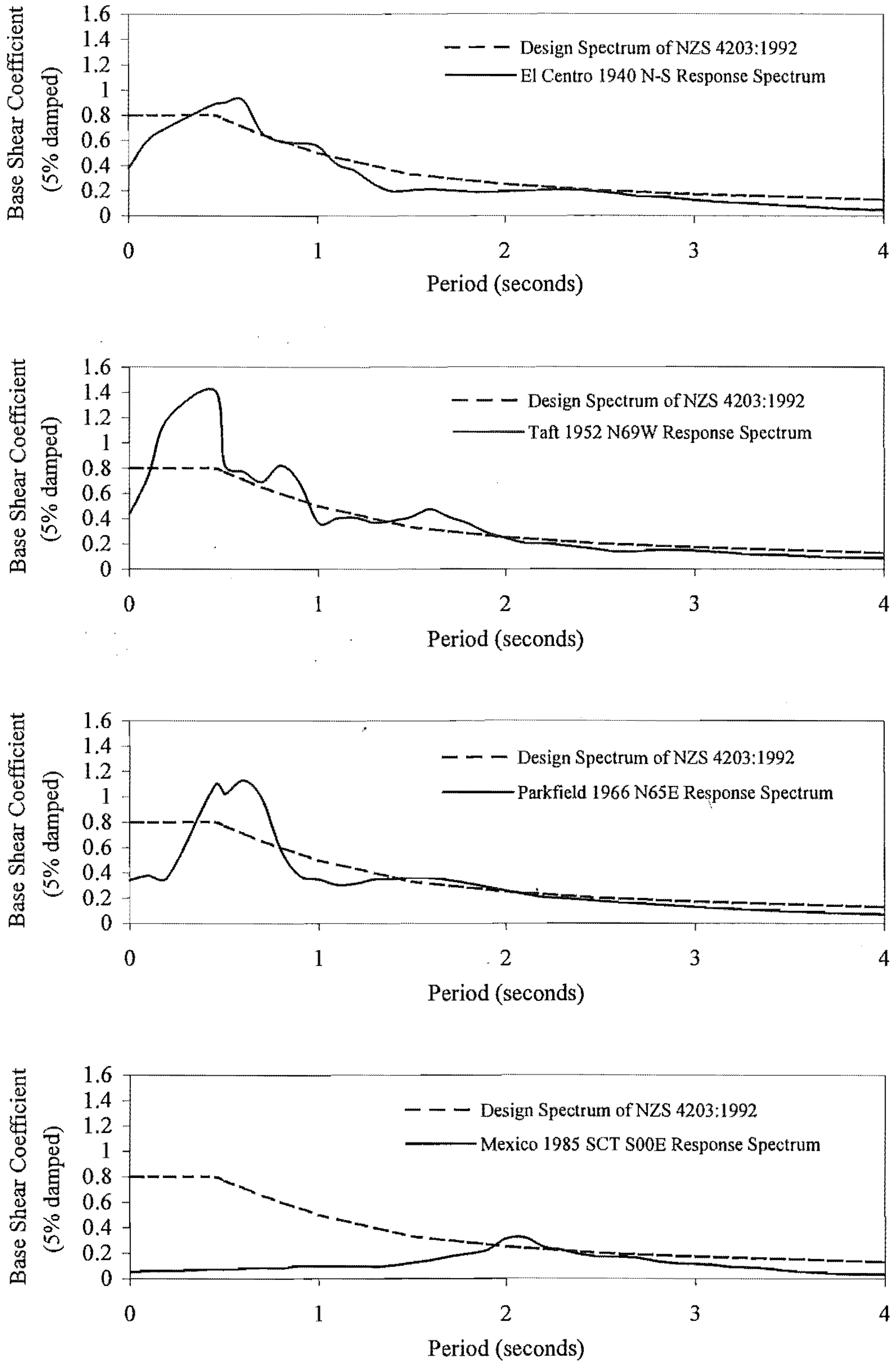
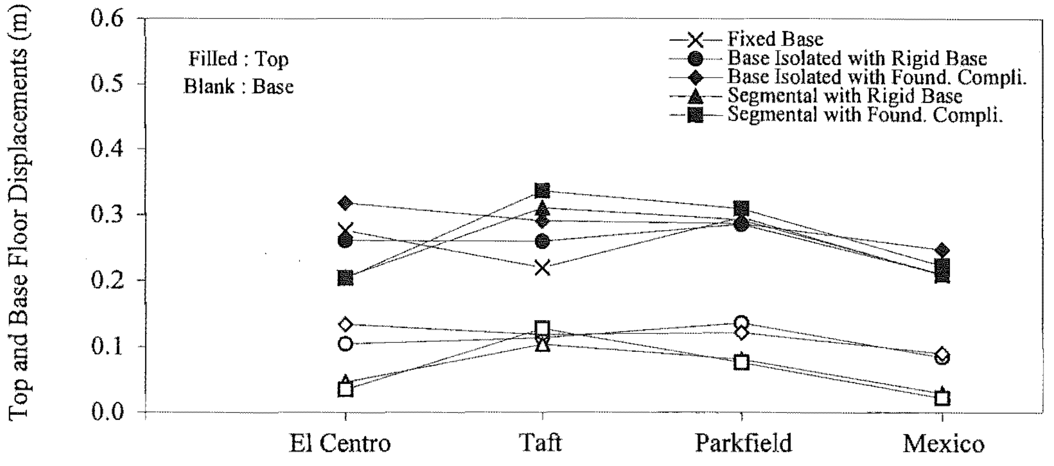
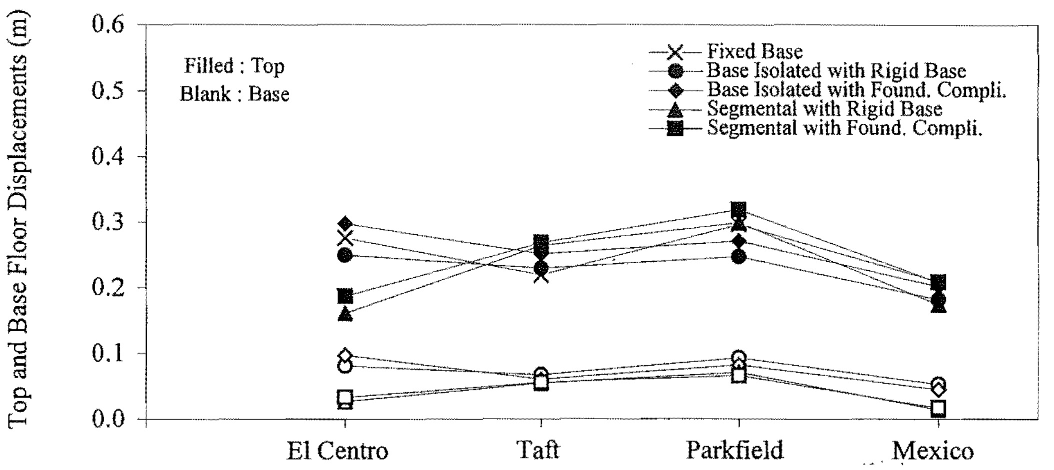


Fig. C.4 Base Shear Coefficients for the Earthquake Response Spectrum and Design Spectrum for Segmental Buildings Designed to NZS 3101:1982



(a) Structures with Elasto-Plastic Isolation Systems



(b) Structures with Bilinear Isolation Systems

Fig. C.5 Top and Base Floor Displacements of Structures
for Different Earthquakes

Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.276	0.219	0.296	0.209
Base Isolated Building with Rigid base	0.157	0.146	0.149	0.126
Base Isolated Building with Foundation Compliance	0.184	0.172	0.164	0.157
Segmental Building with Rigid Base	0.178	0.207	0.211	0.181
Segmental Building with Foundation Compliance	0.181	0.209	0.233	0.201

(a) Buildings Mounted on Elasto-Plastic Isolation Systems

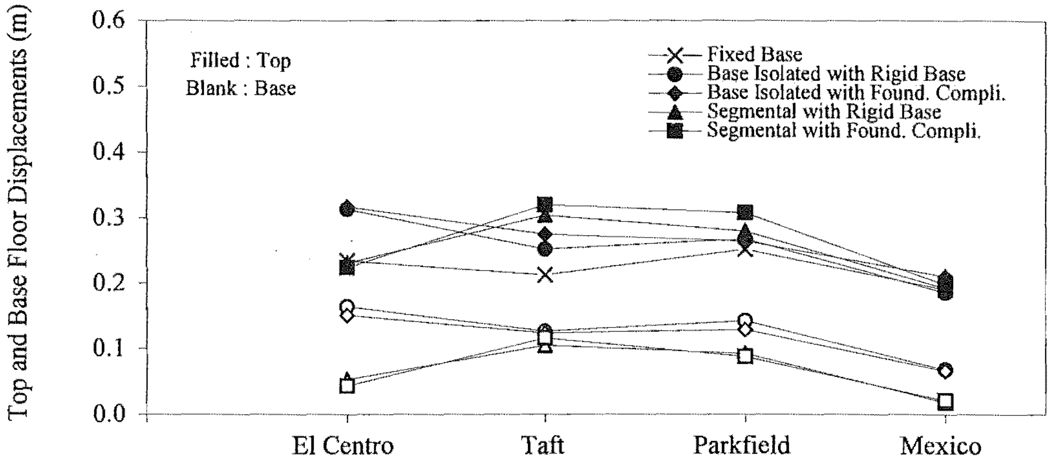
Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.276	0.219	0.296	0.209
Base Isolated Building with Rigid base	0.169	0.162	0.154	0.129
Base Isolated Building with Foundation Compliance	0.201	0.191	0.189	0.157
Segmental Building with Rigid Base	0.141	0.210	0.228	0.160
Segmental Building with Foundation Compliance	0.158	0.213	0.253	0.191

(b) Buildings Mounted on Bilinear Isolation Systems

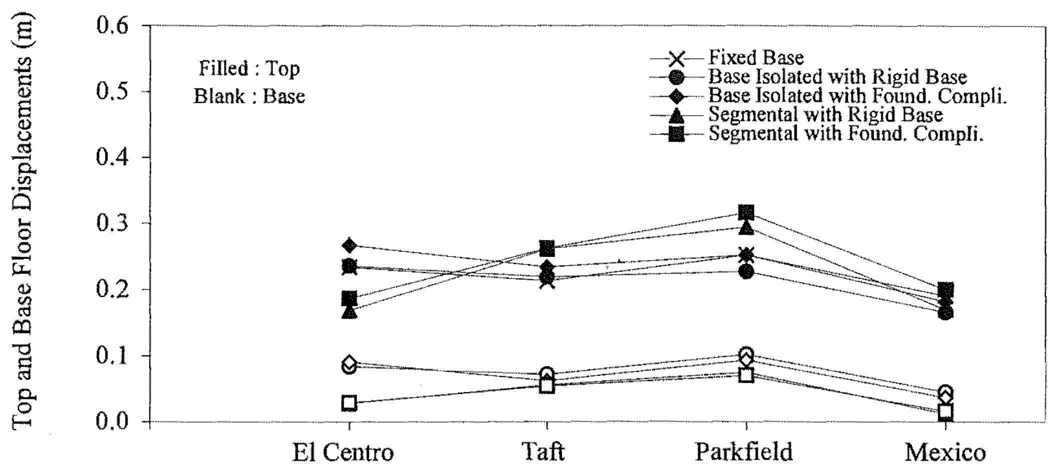
Note:

Top floor displacements of the base isolated and segmental buildings are based on the deduction of base floor displacements.

Table C.1 Top Floor Displacements of Structures for Different Earthquakes



(a) Structures with Elasto-Plastic Isolation Systems



(b) Structures with Bilinear Isolation Systems

Fig. C.6 Top and Base Floor Displacements of Structures with Additional Damping for Different Earthquakes

Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.234	0.213	0.252	0.191
Base Isolated Building with Rigid base	0.149	0.125	0.125	0.118
Base Isolated Building with Foundation Compliance	0.166	0.151	0.136	0.145
Segmental Building with Rigid Base	0.159	0.199	0.187	0.175
Segmental Building with Foundation Compliance	0.169	0.204	0.220	0.177

(a) Buildings Mounted on Elasto-Plastic Isolation Systems

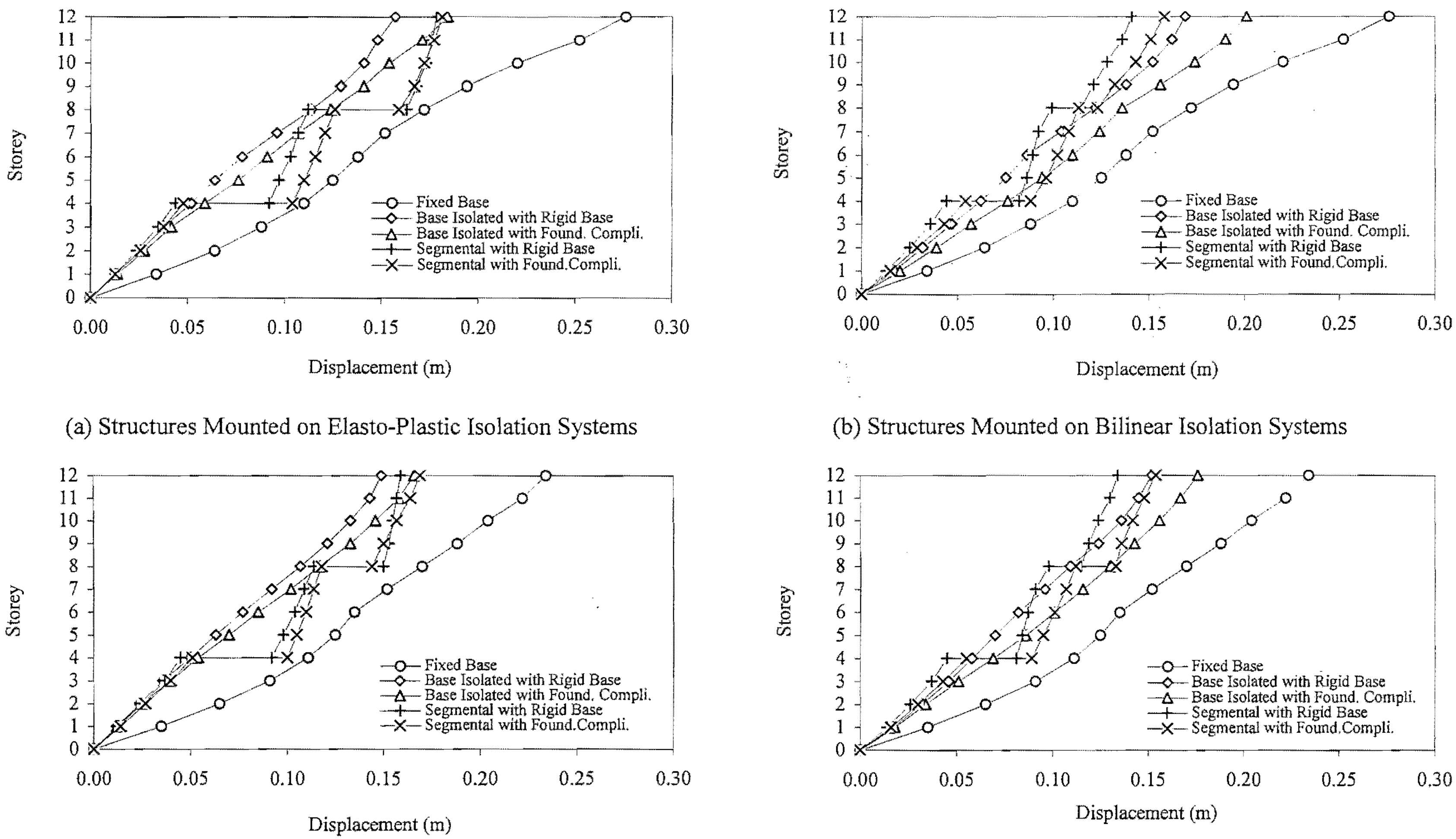
Building Types	Top Floor Displacements (m)			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.234	0.213	0.252	0.191
Base Isolated Building with Rigid base	0.152	0.147	0.125	0.121
Base Isolated Building with Foundation Compliance	0.176	0.172	0.157	0.145
Segmental Building with Rigid Base	0.134	0.205	0.219	0.158
Segmental Building with Foundation Compliance	0.154	0.208	0.246	0.184

(b) Buildings Mounted on Bilinear Isolation Systems

Note:

Top floor displacements of the base isolated and segmental buildings are based on the deduction of base floor displacements.

Table C.2 Top Floor Displacements of Structures with Additional Damping for Different Earthquakes



(a) Structures Mounted on Elasto-Plastic Isolation Systems

(b) Structures Mounted on Bilinear Isolation Systems

(c) Structures with Additional Damping Mounted on Elasto-Plastic Isolation Systems

(d) Structures with Additional Damping Mounted on Bilinear Isolation Systems

Fig. C.7 Comparisons of Displacement with Storey of Different Structures for the El Centro 1940 N-S Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.66%	0.25%	0.36%	0.08%	0.14%
11	0.88%	0.19%	0.47%	0.08%	0.19%
10	0.71%	0.33%	0.36%	0.08%	0.19%
9	0.60%	0.41%	0.47%	0.08%	0.16%
8	0.55%	0.49%	0.47%	0.14%	0.11%
7	0.38%	0.49%	0.44%	0.14%	0.11%
6	0.36%	0.38%	0.41%	0.16%	0.14%
5	0.41%	0.33%	0.47%	0.16%	0.14%
4	0.60%	0.38%	0.47%	0.22%	0.30%
3	0.66%	0.33%	0.38%	0.33%	0.36%
2	0.82%	0.36%	0.38%	0.33%	0.36%
1	0.68%	0.26%	0.28%	0.26%	0.28%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.66%	0.19%	0.30%	0.11%	0.16%
11	0.88%	0.27%	0.44%	0.16%	0.16%
10	0.71%	0.38%	0.49%	0.14%	0.16%
9	0.60%	0.44%	0.55%	0.11%	0.11%
8	0.55%	0.49%	0.33%	0.19%	0.14%
7	0.38%	0.49%	0.38%	0.11%	0.16%
6	0.36%	0.30%	0.44%	0.11%	0.16%
5	0.41%	0.36%	0.49%	0.11%	0.16%
4	0.60%	0.41%	0.52%	0.22%	0.33%
3	0.66%	0.41%	0.49%	0.30%	0.36%
2	0.82%	0.41%	0.52%	0.33%	0.38%
1	0.68%	0.32%	0.40%	0.28%	0.32%

(b) Structures Mounted on Bilinear Isolation Systems

Table C.3 Interstorey Drifts of Structures for the El Centro 1940 N-S Earthquake

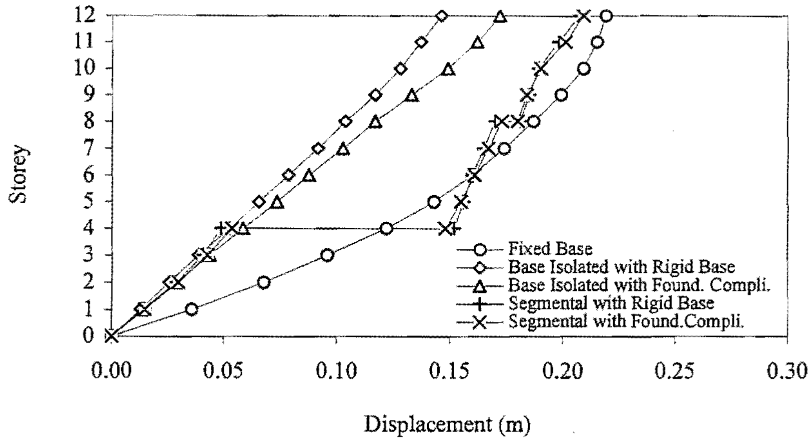
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.16%	0.22%	0.08%	0.11%
11	0.49%	0.27%	0.33%	0.11%	0.14%
10	0.44%	0.33%	0.36%	0.14%	0.14%
9	0.49%	0.38%	0.44%	0.14%	0.22%
8	0.49%	0.41%	0.41%	0.14%	0.14%
7	0.47%	0.41%	0.47%	0.11%	0.14%
6	0.27%	0.38%	0.41%	0.16%	0.16%
5	0.38%	0.36%	0.44%	0.14%	0.16%
4	0.55%	0.38%	0.38%	0.25%	0.27%
3	0.71%	0.33%	0.41%	0.30%	0.33%
2	0.82%	0.33%	0.36%	0.33%	0.36%
1	0.70%	0.24%	0.24%	0.26%	0.26%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

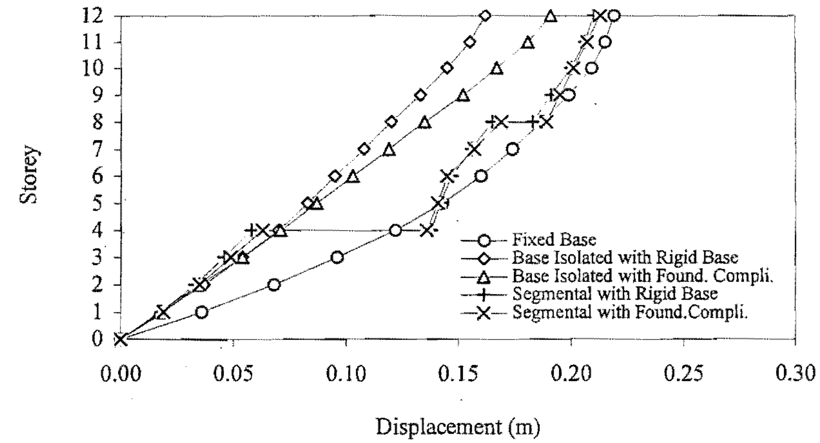
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.19%	0.25%	0.14%	0.19%
11	0.49%	0.25%	0.30%	0.22%	0.22%
10	0.44%	0.33%	0.36%	0.19%	0.30%
9	0.49%	0.41%	0.36%	0.19%	0.25%
8	0.49%	0.36%	0.38%	0.19%	0.14%
7	0.47%	0.38%	0.41%	0.11%	0.16%
6	0.27%	0.33%	0.41%	0.11%	0.16%
5	0.38%	0.33%	0.47%	0.11%	0.22%
4	0.55%	0.33%	0.49%	0.22%	0.30%
3	0.71%	0.38%	0.47%	0.30%	0.38%
2	0.82%	0.41%	0.44%	0.33%	0.38%
1	0.70%	0.34%	0.36%	0.26%	0.30%

(b) Structures Mounted on Bilinear Isolation Systems

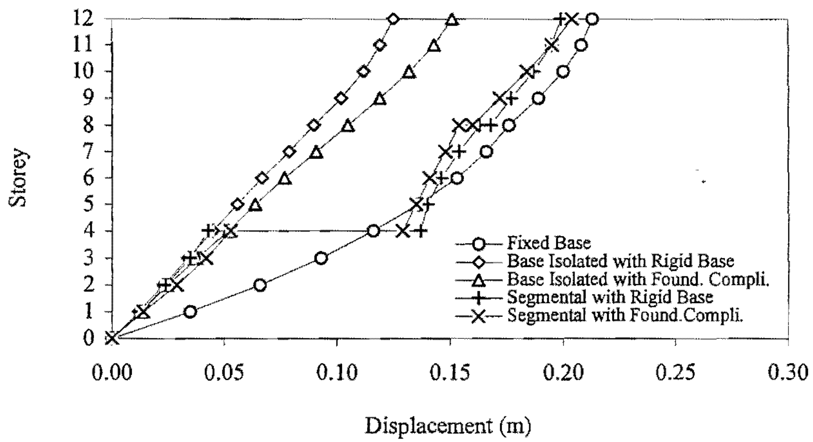
Table C.4 Interstorey Drifts of Structures with Additional Damping for the El Centro 1940 N-S Earthquake



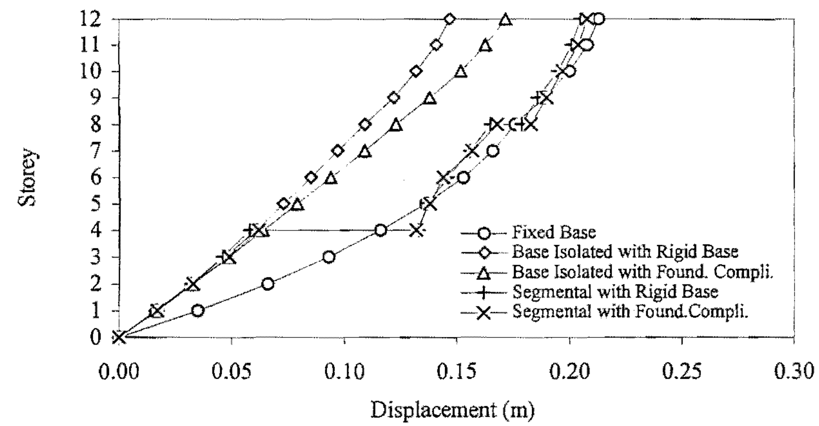
(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems



(c) Structures with Additional Damping Mounted on Elasto-Plastic Isolation Systems



(d) Structures with Additional Damping Mounted on Bilinear Isolation Systems

Fig. C.8 Comparisons of Displacement with Storey of Different Structures for the Taft 1952 N69W Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.11%	0.25%	0.27%	0.25%	0.25%
11	0.16%	0.25%	0.36%	0.25%	0.30%
10	0.27%	0.30%	0.44%	0.11%	0.16%
9	0.33%	0.36%	0.44%	0.11%	0.11%
8	0.36%	0.33%	0.38%	0.14%	0.16%
7	0.38%	0.36%	0.41%	0.14%	0.16%
6	0.47%	0.36%	0.38%	0.11%	0.16%
5	0.58%	0.36%	0.41%	0.11%	0.19%
4	0.71%	0.38%	0.41%	0.22%	0.30%
3	0.77%	0.36%	0.41%	0.33%	0.36%
2	0.88%	0.36%	0.41%	0.38%	0.41%
1	0.72%	0.26%	0.28%	0.30%	0.30%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.11%	0.19%	0.27%	0.11%	0.16%
11	0.16%	0.27%	0.38%	0.16%	0.16%
10	0.27%	0.33%	0.41%	0.25%	0.16%
9	0.33%	0.36%	0.47%	0.22%	0.16%
8	0.36%	0.33%	0.44%	0.25%	0.33%
7	0.38%	0.36%	0.44%	0.25%	0.33%
6	0.47%	0.33%	0.44%	0.14%	0.11%
5	0.58%	0.36%	0.44%	0.11%	0.14%
4	0.71%	0.44%	0.47%	0.33%	0.38%
3	0.77%	0.47%	0.52%	0.36%	0.38%
2	0.88%	0.52%	0.49%	0.41%	0.44%
1	0.72%	0.36%	0.34%	0.36%	0.38%

(b) Structures Mounted on Bilinear Isolation Systems

Table C.5 Interstorey Drifts of Structures for the Taft 1952 N69W Earthquake

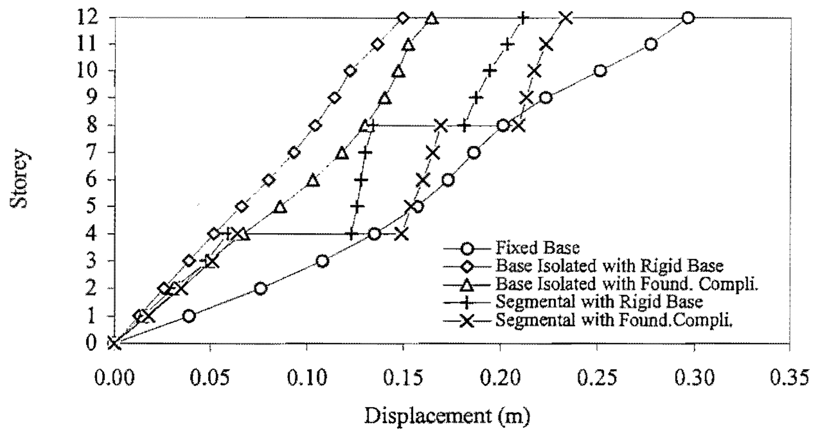
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.16%	0.22%	0.11%	0.25%
11	0.22%	0.19%	0.30%	0.22%	0.30%
10	0.30%	0.27%	0.36%	0.27%	0.33%
9	0.36%	0.33%	0.38%	0.25%	0.33%
8	0.27%	0.30%	0.38%	0.25%	0.16%
7	0.36%	0.33%	0.38%	0.22%	0.19%
6	0.47%	0.30%	0.36%	0.16%	0.16%
5	0.55%	0.30%	0.33%	0.11%	0.16%
4	0.63%	0.30%	0.38%	0.22%	0.25%
3	0.74%	0.30%	0.36%	0.30%	0.36%
2	0.85%	0.30%	0.33%	0.30%	0.41%
1	0.70%	0.24%	0.26%	0.26%	0.28%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

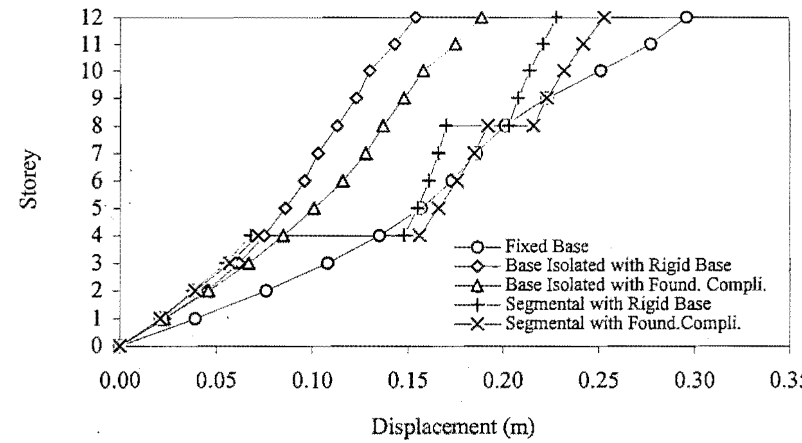
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.14%	0.16%	0.25%	0.11%	0.11%
11	0.22%	0.25%	0.30%	0.19%	0.19%
10	0.30%	0.27%	0.38%	0.25%	0.19%
9	0.36%	0.36%	0.41%	0.19%	0.19%
8	0.27%	0.33%	0.38%	0.25%	0.30%
7	0.36%	0.33%	0.41%	0.27%	0.36%
6	0.47%	0.33%	0.41%	0.25%	0.16%
5	0.55%	0.33%	0.41%	0.11%	0.16%
4	0.63%	0.38%	0.41%	0.33%	0.36%
3	0.74%	0.41%	0.47%	0.38%	0.44%
2	0.85%	0.44%	0.44%	0.41%	0.44%
1	0.70%	0.32%	0.32%	0.34%	0.34%

(b) Structures Mounted on Bilinear Isolation Systems

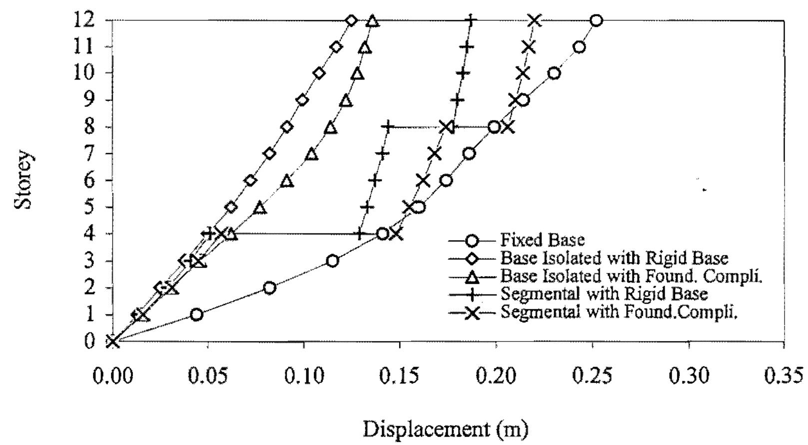
Table C.6 Interstorey Drifts of Structures with Additional Damping for the Taft 1952 N69W Earthquake



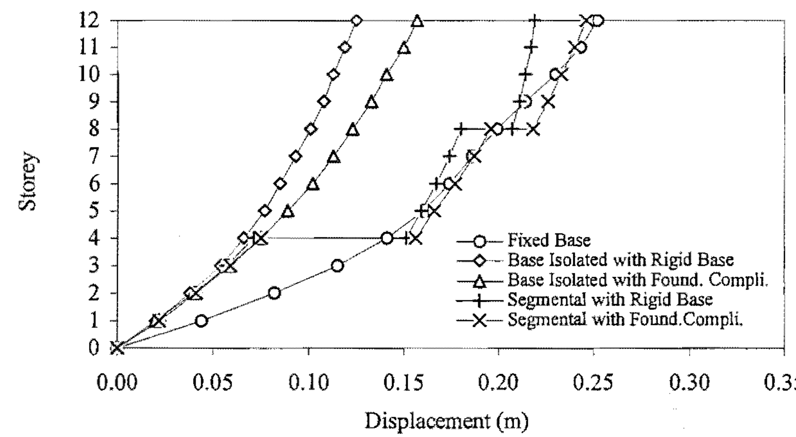
(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems



(c) Structures with Additional Damping Mounted on
Elasto-Plastic Isolation Systems



(d) Structures with Additional Damping Mounted on
Bilinear Isolation Systems

Fig. C.9 Comparisons of Displacement with Storey of Different
Structures for the Parkfield 1966 N65E Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.52%	0.36%	0.33%	0.22%	0.27%
11	0.71%	0.38%	0.14%	0.25%	0.16%
10	0.77%	0.22%	0.19%	0.19%	0.11%
9	0.60%	0.27%	0.27%	0.16%	0.11%
8	0.41%	0.30%	0.33%	0.14%	0.11%
7	0.36%	0.36%	0.41%	0.11%	0.14%
6	0.44%	0.38%	0.47%	0.11%	0.16%
5	0.60%	0.38%	0.52%	0.11%	0.14%
4	0.74%	0.36%	0.49%	0.30%	0.36%
3	0.88%	0.36%	0.49%	0.38%	0.44%
2	1.01%	0.36%	0.44%	0.47%	0.47%
1	0.78%	0.26%	0.30%	0.34%	0.36%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.52%	0.30%	0.38%	0.19%	0.30%
11	0.71%	0.36%	0.47%	0.19%	0.27%
10	0.77%	0.19%	0.27%	0.16%	0.25%
9	0.60%	0.27%	0.30%	0.14%	0.19%
8	0.41%	0.27%	0.25%	0.11%	0.19%
7	0.36%	0.19%	0.33%	0.14%	0.25%
6	0.44%	0.27%	0.41%	0.16%	0.27%
5	0.60%	0.30%	0.44%	0.19%	0.27%
4	0.74%	0.36%	0.49%	0.36%	0.41%
3	0.88%	0.47%	0.58%	0.47%	0.49%
2	1.01%	0.60%	0.63%	0.47%	0.49%
1	0.78%	0.46%	0.46%	0.42%	0.42%

(b) Structures Mounted on Bilinear Isolation Systems

Table C.7 Interstorey Drifts of Structures for the Parkfield 1966 N65E Earthquake

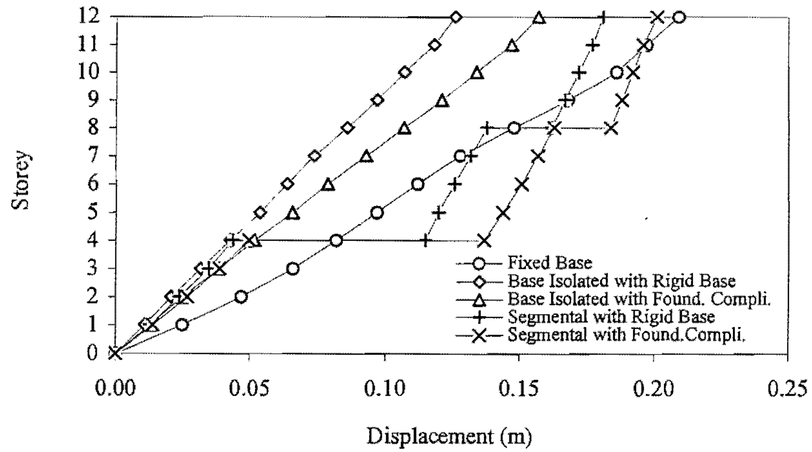
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.25%	0.22%	0.11%	0.08%	0.08%
11	0.36%	0.25%	0.11%	0.08%	0.08%
10	0.44%	0.25%	0.16%	0.08%	0.11%
9	0.41%	0.22%	0.22%	0.08%	0.11%
8	0.36%	0.25%	0.27%	0.08%	0.16%
7	0.33%	0.27%	0.36%	0.11%	0.16%
6	0.38%	0.27%	0.38%	0.11%	0.19%
5	0.52%	0.33%	0.41%	0.11%	0.19%
4	0.71%	0.33%	0.44%	0.27%	0.33%
3	0.90%	0.36%	0.44%	0.33%	0.38%
2	1.04%	0.33%	0.41%	0.38%	0.41%
1	0.88%	0.26%	0.30%	0.30%	0.32%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

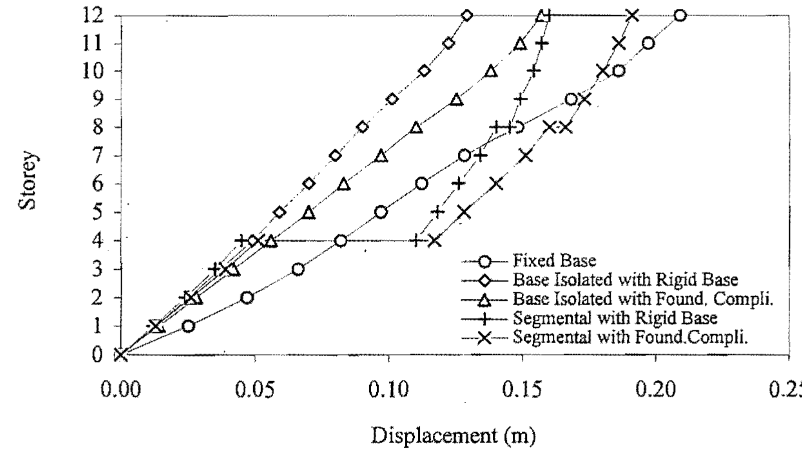
Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.25%	0.16%	0.19%	0.08%	0.16%
11	0.36%	0.16%	0.25%	0.08%	0.19%
10	0.44%	0.14%	0.22%	0.08%	0.19%
9	0.41%	0.19%	0.27%	0.11%	0.22%
8	0.36%	0.22%	0.27%	0.16%	0.25%
7	0.33%	0.22%	0.30%	0.19%	0.27%
6	0.38%	0.22%	0.36%	0.22%	0.30%
5	0.52%	0.30%	0.38%	0.22%	0.27%
4	0.71%	0.33%	0.47%	0.38%	0.44%
3	0.90%	0.44%	0.49%	0.44%	0.49%
2	1.04%	0.49%	0.55%	0.52%	0.52%
1	0.88%	0.40%	0.40%	0.44%	0.44%

(b) Structures Mounted on Bilinear Isolation Systems

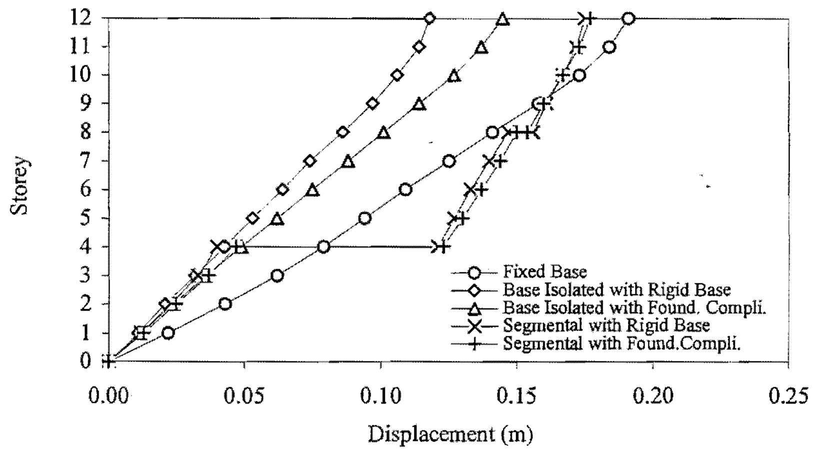
Table C.8 Interstorey Drifts of Structures with Additional Damping for the Parkfield 1966 N65E Earthquake



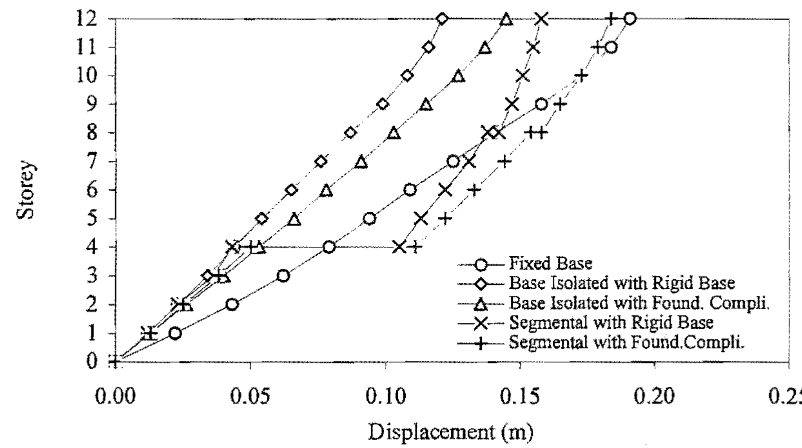
(a) Structures Mounted on Elasto-Plastic Isolation Systems



(b) Structures Mounted on Bilinear Isolation Systems



(c) Structures with Additional Damping Mounted on Elasto-Plastic Isolation Systems



(d) Structures with Additional Damping Mounted on Bilinear Isolation Systems

Fig. C.10 Comparisons of Displacement with Storey of Different Structures for the Mexico 1985 SCT S00E Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.22%	0.27%	0.11%	0.14%
11	0.30%	0.30%	0.36%	0.14%	0.11%
10	0.49%	0.27%	0.36%	0.14%	0.11%
9	0.55%	0.30%	0.38%	0.14%	0.11%
8	0.55%	0.33%	0.38%	0.16%	0.16%
7	0.44%	0.27%	0.38%	0.16%	0.16%
6	0.41%	0.27%	0.36%	0.16%	0.19%
5	0.41%	0.30%	0.38%	0.14%	0.19%
4	0.44%	0.30%	0.36%	0.25%	0.30%
3	0.52%	0.30%	0.38%	0.30%	0.33%
2	0.60%	0.27%	0.33%	0.30%	0.36%
1	0.50%	0.22%	0.26%	0.26%	0.28%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.33%	0.19%	0.22%	0.08%	0.14%
11	0.30%	0.25%	0.30%	0.08%	0.16%
10	0.49%	0.33%	0.36%	0.14%	0.19%
9	0.55%	0.30%	0.41%	0.11%	0.19%
8	0.55%	0.27%	0.36%	0.16%	0.25%
7	0.44%	0.27%	0.38%	0.22%	0.30%
6	0.41%	0.30%	0.36%	0.22%	0.33%
5	0.41%	0.27%	0.38%	0.22%	0.30%
4	0.44%	0.33%	0.38%	0.27%	0.33%
3	0.52%	0.33%	0.38%	0.30%	0.36%
2	0.60%	0.33%	0.38%	0.33%	0.36%
1	0.50%	0.26%	0.28%	0.24%	0.26%

(b) Structures Mounted on Bilinear Isolation Systems

Table C.9 Interstorey Drifts of Structures for the Mexico 1985 SCT S00E Earthquake

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.19%	0.11%	0.22%	0.08%	0.11%
11	0.30%	0.22%	0.27%	0.14%	0.16%
10	0.41%	0.25%	0.36%	0.16%	0.19%
9	0.47%	0.30%	0.36%	0.14%	0.16%
8	0.44%	0.33%	0.36%	0.19%	0.16%
7	0.44%	0.30%	0.36%	0.19%	0.19%
6	0.41%	0.27%	0.36%	0.16%	0.19%
5	0.41%	0.27%	0.36%	0.16%	0.19%
4	0.47%	0.30%	0.36%	0.36%	0.27%
3	0.52%	0.30%	0.33%	0.27%	0.33%
2	0.58%	0.27%	0.33%	0.30%	0.33%
1	0.44%	0.22%	0.24%	0.24%	0.26%

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Storey	Interstorey Drifts				
	Fixed Base	Base Isolated Building with Rigid Base	Base Isolated with Foundation Compliance	Segmental Building with Rigid Base	Segmental Building with Foundation Compliance
12	0.19%	0.14%	0.22%	0.08%	0.14%
11	0.30%	0.22%	0.27%	0.11%	0.16%
10	0.41%	0.25%	0.33%	0.11%	0.22%
9	0.47%	0.33%	0.33%	0.14%	0.19%
8	0.44%	0.30%	0.33%	0.19%	0.27%
7	0.44%	0.30%	0.36%	0.25%	0.30%
6	0.41%	0.30%	0.33%	0.25%	0.30%
5	0.41%	0.27%	0.36%	0.22%	0.30%
4	0.47%	0.27%	0.36%	0.19%	0.33%
3	0.52%	0.30%	0.38%	0.36%	0.36%
2	0.58%	0.30%	0.36%	0.30%	0.33%
1	0.44%	0.24%	0.26%	0.24%	0.26%

(b) Structures Mounted on Bilinear Isolation Systems

Table C.10 Interstorey Drifts of Structures with Additional Damping for the Mexico 1985 SCT S00E Earthquake

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0826	0.0822	0.0780	0.0748
Base Isolated Building with Rigid base	0.0636	0.0671	0.0631	0.0569
Base Isolated Building with Foundation Compliance	0.0634	0.0642	0.0650	0.0561
Segmental Building with Rigid Base	0.0576	0.0636	0.0657	0.0557
Segmental Building with Foundation Compliance	0.0608	0.0634	0.0645	0.0538

(a) Buildings Mounted on Elasto-Plastic Isolation Systems

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0826	0.0822	0.0780	0.0748
Base Isolated Building with Rigid base	0.0774	0.0759	0.0758	0.0669
Base Isolated Building with Foundation Compliance	0.0762	0.0708	0.0752	0.0630
Segmental Building with Rigid Base	0.0624	0.0668	0.0747	0.0538
Segmental Building with Foundation Compliance	0.0654	0.0697	0.0726	0.0529

(b) Buildings Mounted on Bilinear Isolation Systems

Table C.11 Normalized Base Shears of Structures for Different Earthquakes

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0783	0.0769	0.0768	0.0733
Base Isolated Building with Rigid base	0.0575	0.0608	0.0625	0.0542
Base Isolated Building with Foundation Compliance	0.0570	0.0583	0.0642	0.0531
Segmental Building with Rigid Base	0.0562	0.0580	0.0608	0.0519
Segmental Building with Foundation Compliance	0.0533	0.0575	0.0610	0.0520

(a) Buildings Mounted on Elasto-Plastic Isolation Systems

Building Types	Base Shear / Total Weight of Structure			
	El Centro 1940 N-S	Taft 1952 N69W	Parkfield 1966 N65E	Mexico 1985 SCT S00E
Fixed Base Building	0.0783	0.0769	0.0768	0.0733
Base Isolated Building with Rigid base	0.0695	0.0678	0.0708	0.0627
Base Isolated Building with Foundation Compliance	0.0702	0.0664	0.0698	0.0601
Segmental Building with Rigid Base	0.0584	0.0612	0.0650	0.0525
Segmental Building with Foundation Compliance	0.0598	0.0603	0.0638	0.0522

(b) Buildings Mounted on Bilinear Isolation Systems

Table C.12 Normalized Base Shears of Structures with Additional Damping for Different Earthquakes

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	10.84	2.64	2.54	2.98	2.84
	11	15.57	3.97	3.80	2.97	3.65
	10	11.83	2.85	1.98	1.63	1.07
	9	12.65	3.90	2.88	1.58	1.31
	8	6.76	2.27	2.32	< 1.00	< 1.00
	7	6.17	2.70	2.45	1.22	1.10
	6	4.45	2.07	1.53	1.03	< 1.00
	5	4.12	1.59	1.24	< 1.00	< 1.00
	4	3.91	1.61	1.56	< 1.00	< 1.00
	3	4.83	1.47	1.39	< 1.00	< 1.00
	2	5.41	1.41	1.33	1.22	1.11
	1	6.66	1.27	1.22	1.34	1.19
Column Bases	L. Ext.*	1.72	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R. Ext.*	1.74	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	10.84	3.23	2.94	2.68	3.11
	11	15.57	4.02	4.23	3.35	3.82
	10	11.83	3.04	2.32	1.69	1.50
	9	12.65	3.70	3.11	1.48	1.41
	8	6.76	2.50	2.66	< 1.00	< 1.00
	7	6.17	3.06	3.01	< 1.00	1.11
	6	4.45	2.36	2.19	< 1.00	< 1.00
	5	4.12	2.11	1.76	< 1.00	< 1.00
	4	3.91	2.64	2.23	< 1.00	< 1.00
	3	4.83	2.42	2.52	< 1.00	1.04
	2	5.41	2.56	2.14	1.15	1.28
	1	6.66	2.39	2.11	1.56	1.55
Column Bases	L. Ext.*	1.72	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R. Ext.*	1.74	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.13 Maximum Curvature Ductility Demands of Structures for the El Centro 1940 N-S Earthquake

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	5.46	4.32	3.28	2.09	3.14
	11	9.59	3.19	4.50	2.69	3.38
	10	7.64	2.81	2.45	1.34	1.37
	9	7.96	2.72	2.65	1.27	1.24
	8	5.85	2.56	1.75	< 1.00	< 1.00
	7	5.18	1.53	1.50	1.17	1.50
	6	4.25	1.65	< 1.00	< 1.00	< 1.00
	5	5.31	1.45	1.04	< 1.00	< 1.00
	4	4.76	1.35	1.23	< 1.00	< 1.00
	3	5.60	1.40	1.15	1.17	1.19
	2	6.13	1.42	1.20	1.76	1.63
	1	7.74	1.38	1.22	1.97	1.57
Column Bases	L.Ext.*	1.63	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.60	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	5.46	4.53	2.91	2.68	3.89
	11	9.59	3.33	4.21	3.41	3.34
	10	7.64	2.62	2.49	1.06	1.17
	9	7.96	2.59	2.52	1.18	< 1.00
	8	5.85	2.04	1.72	< 1.00	< 1.00
	7	5.18	1.50	1.61	1.47	1.69
	6	4.25	1.37	1.35	1.08	1.21
	5	5.31	1.38	1.55	1.12	< 1.00
	4	4.76	1.59	1.79	< 1.00	< 1.00
	3	5.60	2.22	1.72	1.81	1.45
	2	6.13	2.75	1.93	2.23	1.78
	1	7.74	2.87	1.72	2.58	2.22
Column Bases	L.Ext.*	1.63	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.60	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.14 Maximum Curvature Ductility Demands of Structures for the Taft 1952 N69W Earthquake

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	9.51	4.31	4.45	3.54	4.62
	11	12.43	4.92	4.75	4.13	4.74
	10	10.11	4.27	3.18	1.93	1.45
	9	6.50	4.40	3.25	1.34	1.19
	8	4.39	2.03	1.41	< 1.00	< 1.00
	7	4.10	1.56	1.03	< 1.00	1.38
	6	3.55	1.35	1.20	< 1.00	1.16
	5	4.71	1.72	1.90	< 1.00	< 1.00
	4	5.12	1.75	2.29	< 1.00	< 1.00
	3	5.98	1.56	2.14	1.27	1.30
	2	6.36	1.39	1.93	2.01	1.91
	1	7.56	1.32	1.32	2.37	2.05
Column Bases	L.Ext.*	1.74	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.79	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	9.51	4.64	4.42	3.65	4.46
	11	12.43	5.13	5.00	4.22	4.62
	10	10.11	4.33	3.21	2.01	1.42
	9	6.50	4.40	3.03	1.68	1.16
	8	4.39	1.80	1.30	< 1.00	< 1.00
	7	4.10	1.28	1.01	< 1.00	1.60
	6	3.55	1.29	1.25	1.02	1.25
	5	4.71	1.69	1.99	< 1.00	1.04
	4	5.12	2.00	2.34	< 1.00	< 1.00
	3	5.98	2.49	2.73	2.36	2.14
	2	6.36	3.35	3.20	2.95	2.26
	1	7.56	3.79	3.27	3.24	2.47
Column Bases	L.Ext.*	1.74	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.79	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.15 Maximum Curvature Ductility Demands of Structures for the Parkfield 1966 N65E Earthquake

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	2.42	< 1.00	< 1.00	< 1.00	< 1.00
	11	7.46	1.57	1.84	< 1.00	< 1.00
	10	6.09	1.09	< 1.00	< 1.00	< 1.00
	9	8.53	1.46	1.16	< 1.00	< 1.00
	8	5.29	< 1.00	< 1.00	< 1.00	< 1.00
	7	5.41	< 1.00	< 1.00	< 1.00	< 1.00
	6	4.14	< 1.00	< 1.00	< 1.00	< 1.00
	5	3.92	< 1.00	< 1.00	< 1.00	< 1.00
	4	3.13	< 1.00	< 1.00	< 1.00	< 1.00
	3	3.38	< 1.00	< 1.00	< 1.00	< 1.00
	2	3.91	< 1.00	< 1.00	1.05	< 1.00
	1	5.34	< 1.00	< 1.00	1.16	< 1.00
Column Bases	L.Ext.*	1.21	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.18	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	2.42	< 1.00	< 1.00	< 1.00	< 1.00
	11	7.46	1.65	1.79	< 1.00	< 1.00
	10	6.09	1.19	< 1.00	< 1.00	< 1.00
	9	8.53	1.43	1.09	< 1.00	< 1.00
	8	5.29	< 1.00	< 1.00	< 1.00	< 1.00
	7	5.41	< 1.00	< 1.00	< 1.00	< 1.00
	6	4.14	< 1.00	< 1.00	< 1.00	< 1.00
	5	3.92	< 1.00	< 1.00	< 1.00	< 1.00
	4	3.13	< 1.00	< 1.00	< 1.00	< 1.00
	3	3.38	< 1.00	< 1.00	< 1.00	< 1.00
	2	3.91	1.15	1.10	1.09	< 1.00
	1	5.34	1.27	< 1.00	1.11	< 1.00
Column Bases	L.Ext.*	1.21	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.18	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:
* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.16 Maximum Curvature Ductility Demands of Structures for the Mexico 1985 SCT S00E Earthquake

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	4.67	< 1.00	< 1.00	< 1.00	< 1.00
	11	10.29	1.58	1.80	1.26	1.88
	10	8.40	1.22	< 1.00	< 1.00	< 1.00
	9	9.40	1.86	1.31	< 1.00	< 1.00
	8	5.50	1.32	1.20	< 1.00	< 1.00
	7	5.31	1.62	1.37	< 1.00	< 1.00
	6	4.48	1.36	< 1.00	< 1.00	< 1.00
	5	4.03	1.41	1.04	< 1.00	< 1.00
	4	3.46	1.34	1.12	< 1.00	< 1.00
	3	4.62	1.28	1.02	< 1.00	< 1.00
	2	5.60	1.15	< 1.00	1.07	< 1.00
	1	7.18	< 1.00	< 1.00	1.19	< 1.00
Column Bases	L.Ext.*	1.63	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.61	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	4.67	< 1.00	1.01	< 1.00	< 1.00
	11	10.29	1.84	2.09	1.45	2.00
	10	8.40	1.21	< 1.00	< 1.00	< 1.00
	9	9.40	1.67	1.15	< 1.00	< 1.00
	8	5.50	1.12	1.25	< 1.00	< 1.00
	7	5.31	1.58	1.65	< 1.00	< 1.00
	6	4.48	1.39	1.18	< 1.00	< 1.00
	5	4.03	1.52	1.28	< 1.00	< 1.00
	4	3.46	1.58	1.64	< 1.00	< 1.00
	3	4.62	1.85	1.88	< 1.00	< 1.00
	2	5.60	2.17	1.98	1.01	1.25
	1	7.18	2.32	1.80	1.40	1.31
Column Bases	L.Ext.*	1.63	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.61	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.17 Maximum Curvature Ductility Demands of Structures with Additional Damping for the El Centro 1940 N-S Earthquake

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	2.07	1.85	< 1.00	< 1.00	1.17
	11	6.67	1.66	1.86	1.29	1.28
	10	5.15	1.26	< 1.00	< 1.00	< 1.00
	9	6.58	1.69	1.06	< 1.00	< 1.00
	8	3.46	1.52	< 1.00	< 1.00	< 1.00
	7	4.16	1.12	< 1.00	< 1.00	< 1.00
	6	4.17	< 1.00	< 1.00	< 1.00	< 1.00
	5	4.88	< 1.00	< 1.00	< 1.00	< 1.00
	4	4.23	< 1.00	< 1.00	< 1.00	< 1.00
	3	4.97	< 1.00	< 1.00	1.03	< 1.00
	2	5.80	1.02	< 1.00	1.54	1.40
	1	7.54	1.06	< 1.00	1.46	1.36
Column Bases	L.Ext.*	1.56	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.54	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
Beam Ends	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
	12	2.07	1.82	1.02	< 1.00	1.32
	11	6.67	1.64	1.84	< 1.00	1.27
	10	5.15	1.22	< 1.00	< 1.00	< 1.00
	9	6.58	1.46	< 1.00	< 1.00	< 1.00
	8	3.46	1.26	< 1.00	< 1.00	< 1.00
	7	4.16	< 1.00	< 1.00	< 1.00	< 1.00
	6	4.17	< 1.00	< 1.00	< 1.00	< 1.00
	5	4.88	< 1.00	< 1.00	< 1.00	< 1.00
	4	4.23	1.26	1.15	< 1.00	< 1.00
	3	4.97	1.68	1.30	1.56	1.27
	2	5.80	2.05	1.49	2.14	1.68
	1	7.54	2.26	1.43	2.53	1.91
Column Bases	L.Ext.*	1.56	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.54	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.18 Maximum Curvature Ductility Demands of Structures with Additional Damping for the Taft 1952 N69W Earthquake

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	3.84	1.28	1.09	< 1.00	< 1.00
	11	8.91	2.50	2.36	1.45	2.01
	10	6.86	1.81	1.02	< 1.00	< 1.00
	9	7.56	2.42	1.51	< 1.00	< 1.00
	8	3.82	1.24	< 1.00	< 1.00	< 1.00
	7	3.30	< 1.00	< 1.00	< 1.00	< 1.00
	6	2.88	< 1.00	< 1.00	< 1.00	< 1.00
	5	4.34	< 1.00	< 1.00	< 1.00	< 1.00
	4	4.58	1.15	1.35	< 1.00	< 1.00
	3	5.93	1.28	1.53	< 1.00	< 1.00
	2	6.75	1.26	1.54	1.57	1.51
	1	8.22	1.25	1.19	1.75	1.61
Column Bases	L.Ext.*	1.67	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.69	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	3.84	1.27	1.13	< 1.00	< 1.00
	11	8.91	2.45	2.49	1.60	2.02
	10	6.86	1.79	1.01	< 1.00	< 1.00
	9	7.56	2.43	1.51	< 1.00	< 1.00
	8	3.82	1.21	< 1.00	< 1.00	< 1.00
	7	3.30	< 1.00	< 1.00	< 1.00	1.03
	6	2.88	< 1.00	< 1.00	< 1.00	< 1.00
	5	4.34	< 1.00	< 1.00	< 1.00	< 1.00
	4	4.58	1.24	1.45	< 1.00	< 1.00
	3	5.93	1.74	1.92	2.25	2.12
	2	6.75	2.57	2.36	3.16	2.40
	1	8.22	3.28	2.57	3.47	2.80
Column Bases	L.Ext.*	1.67	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	1.69	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.19 Maximum Curvature Ductility Demands of Structures with Additional Damping for the Parkfield 1966 N65E Earthquake

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	1.98	< 1.00	< 1.00	< 1.00	< 1.00
	11	6.54	< 1.00	< 1.00	< 1.00	< 1.00
	10	4.69	< 1.00	< 1.00	< 1.00	< 1.00
	9	6.78	< 1.00	< 1.00	< 1.00	< 1.00
	8	4.09	< 1.00	< 1.00	< 1.00	< 1.00
	7	4.39	< 1.00	< 1.00	< 1.00	< 1.00
	6	3.52	< 1.00	< 1.00	< 1.00	< 1.00
	5	3.81	< 1.00	< 1.00	< 1.00	< 1.00
	4	3.11	< 1.00	< 1.00	< 1.00	< 1.00
	3	3.48	< 1.00	< 1.00	< 1.00	< 1.00
	2	3.89	< 1.00	< 1.00	< 1.00	< 1.00
	1	4.85	< 1.00	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Types		Maximum Curvature Ductility Demands				
	Storey	Fixed Base	Base Isolated with Rigid Base	Base Isolated with Found. Compliance	Segmental with Rigid Base	Segmental with Found. Compliance
Beam Ends	12	1.98	< 1.00	< 1.00	< 1.00	< 1.00
	11	6.54	< 1.00	< 1.00	< 1.00	< 1.00
	10	4.69	< 1.00	< 1.00	< 1.00	< 1.00
	9	6.78	< 1.00	< 1.00	< 1.00	< 1.00
	8	4.09	< 1.00	< 1.00	< 1.00	< 1.00
	7	4.39	< 1.00	< 1.00	< 1.00	< 1.00
	6	3.52	< 1.00	< 1.00	< 1.00	< 1.00
	5	3.81	< 1.00	< 1.00	< 1.00	< 1.00
	4	3.11	< 1.00	< 1.00	< 1.00	< 1.00
	3	3.48	< 1.00	< 1.00	< 1.00	< 1.00
	2	3.89	< 1.00	< 1.00	< 1.00	< 1.00
	1	4.85	1.03	< 1.00	< 1.00	< 1.00
Column Bases	L.Ext.*	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	Inter.**	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00
	R.Ext.*	< 1.00	< 1.00	< 1.00	< 1.00	< 1.00

(b) Structures Mounted on Bilinear Isolation Systems

Note:

* L.Ext. and R.Ext. are External Columns on Left and Right Sides respectively.
** Inter. is Internal Column.

Table C.20 Maximum Curvature Ductility Demands of Structures with Additional Damping for the Mexico 1985 SCT S00E Earthquake

Earthquake Records	Displacement of Isolation System (m)			
	Base Isolated Building	Segmental Building		
	Base Floor	Base Floor	Middle Segment	Top Segment
El Centro 1940 N-S	0.104	0.046	0.059	0.067
Taft 1952 N69W	0.113	0.103	0.108	0.068
Parkfield 1966 N65E	0.136	0.081	0.086	0.062
Mexico 1985 SCT S00E	0.084	0.029	0.080	0.038

(a) Structures Mounted on Elasto-Plastic Isolation Systems

Earthquake Records	Displacement of Isolation System (m)			
	Base Isolated Building	Segmental Building		
	Base Floor	Base Floor	Middle Segment	Top Segment
El Centro 1940 N-S	0.081	0.027	0.044	0.037
Taft 1952 N69W	0.067	0.054	0.057	0.054
Parkfield 1966 N65E	0.093	0.071	0.088	0.039
Mexico 1985 SCT S00E	0.053	0.014	0.048	0.011

(b) Structures Mounted on Bilinear Isolation Systems

Table C.21 Displacements of Isolation Systems for Structures Designed to NZS 3101:1982 for the Four Earthquake Records

APPENDIX D

INPUT DATA FOR COMPUTER ANALYSES

Frame 1. 6-Storey 2 Bays

Section Properties

Member	Level	Dimension (m)	Axial Area (m^2)	Shear Area (m^2)	Moment of Inertia (m^4)	Plastic Hinge Length (m)
Beam	1-3 4-6	0.60 x 0.35 0.55 x 0.35	0.1704 0.1617	0.1050 0.0963	0.005984 0.004635	0.300 0.275
External Column	1-3 4-6	0.50 x 0.45 0.45 x 0.45	0.1688 0.1519	0.1688 0.1519	0.003516 0.002563	0.250 0.225
Internal Column	1-3 4-6	0.55 x 0.55 0.50 x 0.50	0.2269 0.1875	0.2269 0.1875	0.005719 0.003906	0.275 0.250

Column Yield Interaction Data

Column	P_{yc} (KN)	P_B (KN)	M_B (KNm)	M_{1B} (KNm)	M_{2B} (KNm)	M_O (KNm)	P_{yt} (KN)
External	-6290	-3690	435	519	423	197	934
Internal	-8454	-5010	645	775	635	300	1255

Beam Yield Moment (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	M_3 (KNm)	M_4 (KNm)
1-3	262	-262	232	-232
4	173	-184	155	-155
5	115	-131	119	-115
6	115	-115	115	-115

M_1, M_2 and M_3, M_4 are yield moments for the left and right ends respectively.
Yield moments for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Initial Fixed End Moments and Shears (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	V_1 (KN)	V_2 (KN)
1-3	-41.25	40.04	-48.33	48.33
4-6	-41.68	40.05	-47.33	47.33

Initial conditions for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Nodal Loads and Masses

Level		Weight (KN)		Nodal Loads (KN)	
		External Node	Internal Node	External Node	Internal Node
Ground	Base Isolated Fixed Base	134 10	219 13	-85.34 -20.00	-122.50 -26.00
1		134	219	-85.34	-122.50
2		134	219	-85.34	-122.50
3		133	217	-83.34	-118.50
4		129	211	-81.33	-116.50
5		129	211	-81.33	-116.50
6		120	200	-63.33	-94.50

Nodal loads and masses are based on Dead Load plus 1/3 Live Load.
The nodal loads correspond to the gravity load carried by the frame apart from the member loads.

Frame 2. 12-Storey 2 Bays, Fixed Base Building
(Designed to NZS 3101:1982 and NZS 4203:1992)

Section Properties

Member	Level	Dimension (m)	Axial Area (m^2)	Shear Area (m^2)	Moment of Inertia (m^4)	Plastic Hinge Length (m)
Beam	1-6 7-8 9-12	0.900 x 0.400 0.850 x 0.400 0.800 x 0.400	0.4872 0.4772 0.4672	0.1800 0.1700 0.1600	0.02382 0.02017 0.01689	0.450 0.425 0.400
External Column	1-6 7-8 9-12	0.775 x 0.500 0.750 x 0.500 0.650 x 0.500	0.2906 0.2813 0.2438	0.2906 0.2813 0.2438	0.01455 0.01318 0.00855	0.388 0.375 0.325
Internal Column	1-6 7-8 9-12	0.800 x 0.800 0.725 x 0.725 0.675 x 0.675	0.4800 0.3942 0.3417	0.4800 0.3942 0.3417	0.02560 0.01727 0.01297	0.400 0.363 0.338

Column Yield Interaction Data

Column	P_{yc} (KN)	P_B (KN)	M_B (KNm)	M_{1B} (KNm)	M_{2B} (KNm)	M_O (KNm)	P_{yt} (KN)
External	-11152	-6075	1338	1531	1263	665	1930
Internal	-17888	-10920	1986	1986	2450	2038	2656

Beam Yield Moment (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	M_3 (KNm)	M_4 (KNm)
1	976	-976	893	-893
2-4	1142	-1142	1047	-1047
5-6	988	-988	887	-887
7-8	762	-833	714	-714
9-10	559	-631	547	-464
11	307	-369	381	-307
12	307	-307	307	-307

M_1 , M_2 and M_3 , M_4 are yield moments for the left and right ends respectively.
Yield moments for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Initial Fixed End Moments and Shears (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	V_1 (KN)	V_2 (KN)
1-6	-187.8	186.3	-135.8	135.8
7-8	-188.4	186.7	-133.4	133.4
9-12	-188.8	187.2	-131.1	131.1

Initial conditions for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Nodal Loads and Masses

Level	Weight (KN)		Nodal Loads (KN)	
	External Node	Internal Node	External Node	Internal Node
Ground	13	19	-25.0	-37.0
1	434	757	-298.5	-485.1
2	434	757	-298.5	-485.1
3	434	757	-298.5	-485.1
4	434	757	-298.5	-485.1
5	434	757	-298.5	-485.1
6	434	755	-298.5	-485.1
7	427	743	-293.9	-475.9
8	427	743	-293.9	-475.9
9	420	731	-289.2	-468.5
10	420	731	-289.2	-468.5
11	420	731	-289.2	-468.5
12	409	717	-266.2	-439.5

Nodal loads and masses are based on Dead Load plus 1/3 Live Load.
The nodal loads correspond to the gravity load carried by the frame apart from the member loads.

Frame 3. 12-Storey 2 Bays, Base Isolated and Segmental Buildings (Designed to NZS 3101:1982 and NZS 4203:1992)

Section Properties

Member	Level	Dimension (m)	Axial Area (m^2)	Shear Area (m^2)	Moment of Inertia (m^4)	Plastic Hinge Length (m)
Beam	1-6 7-8 9-12	0.900 x 0.400 0.850 x 0.400 0.800 x 0.400	0.4872 0.4772 0.4672	0.1800 0.1700 0.1600	0.02382 0.02017 0.01689	0.450 0.425 0.400
External Column	1-6 7-8 9-12	0.775 x 0.500 0.750 x 0.500 0.650 x 0.500	0.2906 0.2813 0.2438	0.2906 0.2813 0.2438	0.01455 0.01318 0.00855	0.388 0.375 0.325
Internal Column	1-6 7-8 9-12	0.800 x 0.800 0.725 x 0.725 0.675 x 0.675	0.4800 0.3942 0.3417	0.4800 0.3942 0.3417	0.02560 0.01727 0.01297	0.400 0.363 0.338

Column Yield Interaction Data

Column	P_{yc} (KN)	P_B (KN)	M_B (KNm)	M_{1B} (KNm)	M_{2B} (KNm)	M_O (KNm)	P_{yt} (KN)
External	-11858	-4638	1892	1595	1838	1108	3139
Internal	-18432	-7718	2949	2396	2857	1567	4205

Beam Yield Moment (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	M_3 (KNm)	M_4 (KNm)
1	1366	-1366	1250	-1250
2-4	1370	-1370	1256	-1256
5-6	1383	-1383	1242	-1242
7-8	1143	-1250	1071	-1071
9-10	1006	-1136	985	-835
11	614	-738	762	-614
12	430	-430	430	-430

M_1, M_2 and M_3, M_4 are yield moments for the left and right ends respectively.
Yield moments for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Initial Fixed End Moments and Shears (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	V_1 (KN)	V_2 (KN)
1-6	-187.8	186.3	-135.8	135.8
7-8	-188.4	186.7	-133.4	133.4
9-12	-188.8	187.2	-131.1	131.1

Initial conditions for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Nodal Loads and Masses (For Base Isolated Building)

Level	Weight (KN)		Nodal Loads (KN)	
	External Node	Internal Node	External Node	Internal Node
Ground	434	757	-298.5	-485.1
1	434	757	-298.5	-485.1
2	434	757	-298.5	-485.1
3	434	757	-298.5	-485.1
4	434	757	-298.5	-485.1
5	434	757	-298.5	-485.1
6	434	755	-298.5	-485.1
7	427	743	-293.9	-475.9
8	427	743	-293.9	-475.9
9	420	731	-289.2	-468.5
10	420	731	-289.2	-468.5
11	420	731	-289.2	-468.5
12	409	717	-266.2	-439.5

<u>Nodal Loads and Masses</u> (For Segmental Building)

Level	Weight (KN)		Nodal Loads (KN)	
	External Node	Internal Node	External Node	Internal Node
Ground	434	757	-298.5	-485.1
1	434	757	-298.5	-485.1
2	434	757	-298.5	-485.1
3	434	757	-298.5	-485.1
4	434	757	-298.5	-485.1
4A	434	757	-298.5	-485.1
5	434	757	-298.5	-485.1
6	434	755	-298.5	-485.1
7	427	743	-293.9	-475.9
8	427	743	-293.9	-475.9
8A	427	743	-293.9	-475.9
9	420	731	-289.2	-468.5
10	420	731	-289.2	-468.5
11	420	731	-289.2	-468.5
12	409	717	-266.2	-439.5

Nodal loads and masses are based on Dead Load plus 1/3 Live Load.
The nodal loads correspond to the gravity load carried by the frame apart from the member loads.

Frame 4. 12-Storey 2 Bays, Fixed Base Building (Designed to NZS 3101:1995 and NZS 4203:1992)

Section Properties

Member	Level	Dimension (m)	Axial Area (m^2)	Shear Area(m^2)	Moment of Inertia (m^4)	Plastic Hinge Length (m)
Beam	1	0.80 x 0.40	0.2570	0.2570	0.00884	0.400
	2-4	0.80 x 0.40	0.2570	0.2570	0.00884	0.400
	5-8	0.80 x 0.40	0.2570	0.2570	0.00884	0.400
	9-12	0.80 x 0.30	0.1927	0.1927	0.00663	0.400
External Column	1	0.65 x 0.65	0.3929	0.3929	0.01190	0.325
	2-4	0.60 x 0.60	0.3600	0.3600	0.01080	0.300
	5-7	0.60 x 0.60	0.3600	0.3600	0.01080	0.300
	8-10	0.60 x 0.60	0.3600	0.3600	0.01080	0.300
	11-12	0.60 x 0.60	0.3600	0.3600	0.01080	0.300
Internal Column	1	0.75 x 0.75	0.5231	0.5231	0.02109	0.375
	2-4	0.70 x 0.70	0.4900	0.4900	0.02000	0.350
	5-7	0.70 x 0.70	0.4900	0.4900	0.02000	0.350
	8-10	0.70 x 0.70	0.4900	0.4900	0.02000	0.350
	11-12	0.70 x 0.70	0.4900	0.4900	0.02000	0.350

Column Yield Interaction Data

Column	P_{yc} (KN)	P_B (KN)	M_B (KNm)	M_{1B} (KNm)	M_{2B} (KNm)	M_O (KNm)	P_{yt} (KN)
External	-9662	-7284	755	1027	917	420	1478
Internal	-14962	-7425	1446	1932	1780	785	3206

Beam Yield Moment (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	M_3 (KNm)	M_4 (KNm)
1-4	652	-569	570	-717
5-8	643	-575	576	-705
9-12	514	-446	447	-577

M_1 , M_2 and M_3 , M_4 are yield moments for the left and right ends respectively.
Yield moments for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Initial Fixed End Moments and Shears (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	V_1 (KN)	V_2 (KN)
1-9	-252.5	252.5	-142.9	142.9
10-12	-236.4	236.4	-132.5	132.5

Initial conditions for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Nodal Loads and Masses

Level	Weight (KN)		Nodal Loads (KN)	
	External Node	Internal Node	External Node	Internal Node
Ground	22	38	-22	-38
1	346	584	-346	-584
2	340	571	-340	-571
3	340	571	-340	-571
4	337	568	-337	-568
5	335	565	-335	-565
6	335	565	-335	-565
7	333	562	-333	-562
8	331	559	-331	-559
9	331	559	-331	-559
10	329	556	-329	-556
11	327	554	-327	-554
12	232	386	-232	-386

Nodal loads and masses are based on Dead Load plus 1/3 Live Load.
The nodal loads correspond to the gravity load carried by the frame apart from the member loads.

Frame 5. 12-Storey 2 Bays, Base Isolated and Segmental Buildings (Designed to NZS 3101:1995 and NZS 4203:1992)

Section Properties

Member	Level	Dimension (m)	Axial Area (m^2)	Shear Area(m^2)	Moment of Inertia (m^4)	Plastic Hinge Length (m)
Beam	1 2-4 5-8 9-12	0.80 x 0.40 0.80 x 0.40 0.80 x 0.40 0.80 x 0.30	0.2570 0.2570 0.2570 0.1927	0.2570 0.2570 0.2570 0.1927	0.00884 0.00884 0.00884 0.00663	0.400 0.400 0.400 0.400
External Column	1 2-4 5-7 8-10 11-12	0.65 x 0.65 0.60 x 0.60 0.60 x 0.60 0.60 x 0.60 0.60 x 0.60	0.3929 0.3600 0.3600 0.3600 0.3600	0.3929 0.3600 0.3600 0.3600 0.3600	0.01190 0.01080 0.01080 0.01080 0.01080	0.325 0.300 0.300 0.300 0.300
Internal Column	1 2-4 5-7 8-10 11-12	0.75 x 0.75 0.70 x 0.70 0.70 x 0.70 0.70 x 0.70 0.70 x 0.70	0.5231 0.4900 0.4900 0.4900 0.4900	0.5231 0.4900 0.4900 0.4900 0.4900	0.02109 0.02000 0.02000 0.02000 0.02000	0.375 0.350 0.350 0.350 0.350

Column Yield Interaction Data

Column	P_{yc} (KN)	P_B (KN)	M_B (KNm)	M_{1B} (KNm)	M_{2B} (KNm)	M_O (KNm)	P_{yt} (KN)
External	-11712	-5019	1434	1112	1344	680	2167
Internal	-15693	-6733	2202	1708	2088	1063	2936

Beam Yield Moment (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	M_3 (KNm)	M_4 (KNm)
1-4	978	-854	855	-1075
5-6	965	-862	863	-1057
7-8	834	-748	749	-916
9-10	617	-535	536	-692
11-12	565	-491	492	-635

M_1 , M_2 and M_3 , M_4 are yield moments for the left and right ends respectively.
Yield moments for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Initial Fixed End Moments and Shears (Left Bay Beam)

Level	M_1 (KNm)	M_2 (KNm)	V_1 (KN)	V_2 (KN)
1-9	-252.5	252.5	-142.9	142.9
10-12	-236.4	236.4	-132.5	132.5

Initial conditions for the right bay are symmetric.
Positive moment is clockwise on the end of the member.

Nodal Loads and Masses (For Base Isolated Building)

Level	Weight (KN)		Nodal Loads (KN)	
	External Node	Internal Node	External Node	Internal Node
Ground	346	584	-346	-584
1	346	584	-346	-584
2	340	571	-340	-571
3	340	571	-340	-571
4	337	568	-337	-568
5	335	565	-335	-565
6	335	565	-335	-565
7	333	562	-333	-562
8	331	559	-331	-559
9	331	559	-331	-559
10	329	556	-329	-556
11	327	554	-327	-554
12	232	386	-232	-386

Nodal Loads and Masses (For Segmental Building)

Level	Weight (KN)		Nodal Loads (KN)	
	External Node	Internal Node	External Node	Internal Node
Ground	346	584	-346	-584
1	346	584	-346	-584
2	340	571	-340	-571
3	340	571	-340	-571
4	337	568	-337	-568
4A	337	568	-337	-568
5	335	565	-335	-565
6	335	565	-335	-565
7	333	562	-333	-562
8	331	559	-331	-559
8A	331	559	-331	-559
9	331	559	-331	-559
10	329	556	-329	-556
11	327	554	-327	-554
12	232	386	-232	-386

Nodal loads and masses are based on Dead Load plus 1/3 Live Load.
The nodal loads correspond to the gravity load carried by the frame apart from the member loads.

Equivalent Spring and Dashpot Coefficients

Shear Modulus	Foundation	Equivalent Spring and Dashpot Coefficients			
		Vert. Spring (KN/m)	Hor. Spring (KN/m)	Vert. Damping (KN/m/sec)	Hor. Damping (KN/m/sec)
G = 420 (Kg/cm^2)	Ext. Column	308795.35	368176.56	15899.39	13609.67
	Int. Column	412451.64	446707.87	26301.82	20428.69

APPENDIX E

UNIFORM MODELS

Uniform 12-Storey Base Isolated and Segmental Buildings (Designed to NZS 3101:1982 and NZS 4203:1992)

Section Properties

Modulus of Elasticity = 2.5×10^7 KPa

1. Beam (from base floor to 12 floor)

   Dimension (m) : 0.900 x 0.400
   Moment of Inertia (m^4) : 0.02430

2. Column

Member	Level	Height (m)	Dimension (m)	Moment of Inertia (m^4)	Stiffness (KN/m)
External	1	5.00	0.850 x 0.575	0.02943	70630
	2-12	3.65	0.650 x 0.500	0.01144	70630
Internal	1	5.00	0.900 x 0.750	0.04556	109300
	2-12	3.65	0.750 x 0.500	0.01758	109300

3. Joint Mass for frame of each floor

   External Joint (KN) : 422 KN
   Internal Joint (KN) : 757 KN

4. Bass Mass

   Base Isolated Building : The 7.69 % of the total mass (including base mass) of the structure.
   Segmental Building : The 6.67% of the total mass (including base mass) of the structure.

5. Floor

   The mass of each floor (including base floor) is 1601 KN.
   The column stiffness of each floor is 250560 KN/m.

Uniform 12-Storey Base Isolated and Segmental Buildings
(Designed to NZS 3101:1995 and NZS 4203:1992)

Section Properties

Modulus of Elasticity = 2.5 x 10^7 KPa

1. Beam (from base floor to 12 floor)

   Dimension (m) : 0.800 x 0.400
   Moment of Inertia (m^4) : 0.01706

2. Column

Member	Level	Height (m)	Dimension (m)	Moment of Inertia (m^4)	Stiffness (KN/m)
External	1	5.00	0.800 x 0.650	0.02776	66630
	2-12	3.65	0.600 x 0.600	0.01080	66630
Internal	1	5.00	0.850 x 0.750	0.03825	91800
	2-12	3.65	0.650 x 0.650	0.01488	91800

3. Joint Mass for Frame of each Floor

   External Joint (KN) : 312 KN
   Internal Joint (KN) : 584 KN

4. Bass Mass

   Base Isolated Building : The 7.69 % of the total mass (including base mass) of the structure.
   Segmental Building : The 6.67% of the total mass (including base mass) of the structure.

5. Floor

   The mass of each floor (including base floor) is 1208 KN.
   The column stiffness of each floor is 225060 KN/m.